Appendix E

Flood Control Documentation

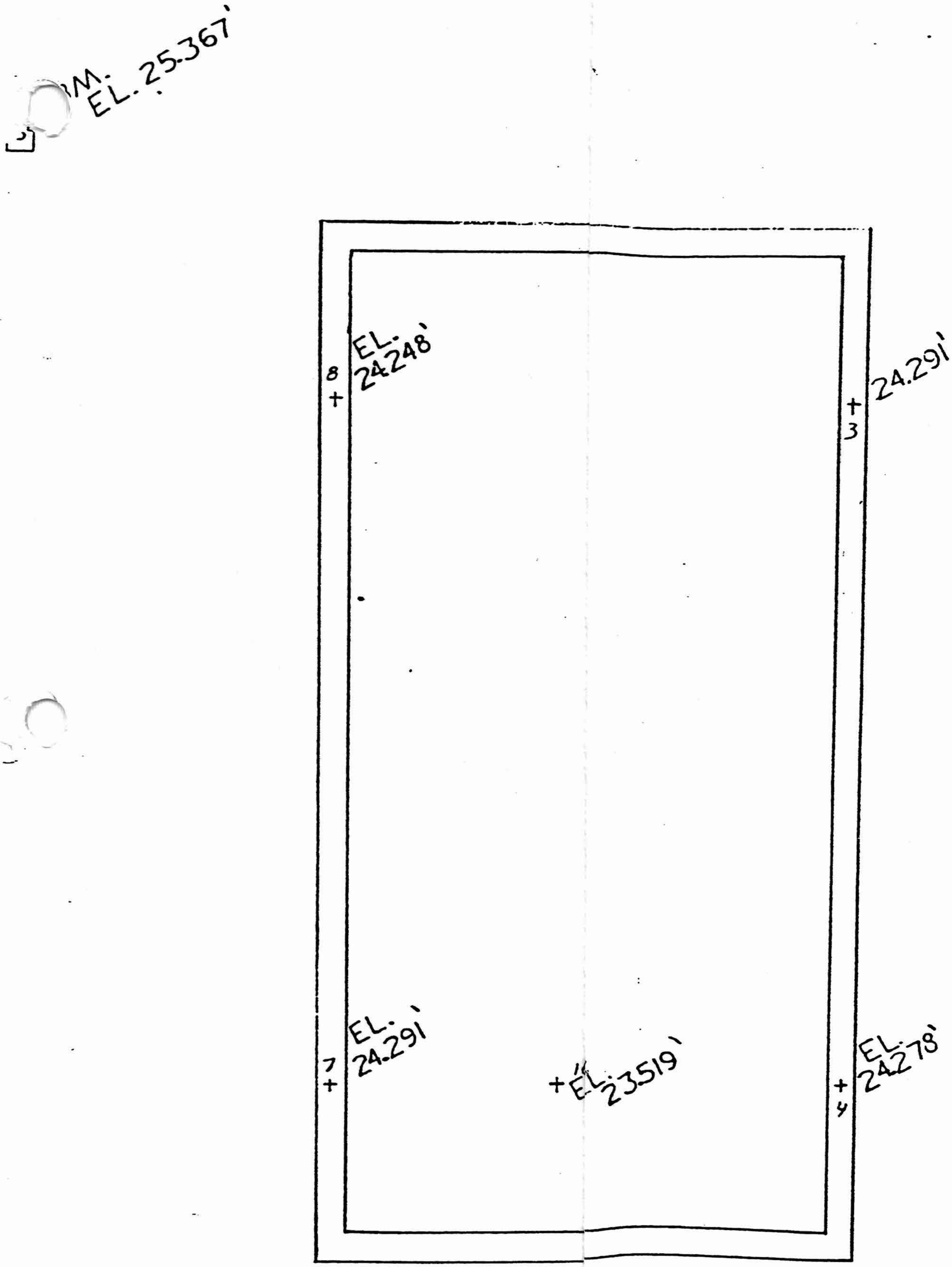
APPENDIX E

FLOOD CONTROL DATA

Handex®

<u>Floodproofing and Flood Protection Measures</u>
SURVEY DATA FOR THE HWSB

Handex®



B.M. EL. 25.367'
8
EL. 24.248'
3
24.291'
7
EL. 24.291'
EL. 23.519'
4
EL. 24.278'

PT.	+	HI	−	ELv.	Descrip.
B.M.	2.481'	27.848'		25.367'	Light Base
⊙1			3.624'	24.224'	
⊙2			3.620'	24.228'	
⊙3			3.557'	24.291'	
⊙4			3.570'	24.278'	
⊙5			3.607'	24.241'	
⊙6			3.606'	24.242'	
⊙7			3.557'	24.291'	
⊙8			3.564'	24.284'	
⊙9			4.320'	23.528'	
⊙10			4.328'	23.520'	
⊙11			4.286'	23.562'	
⊙12			4.329'	23.519'	
⊙13			4.329'	23.519'	
⊙14			4.320'	23.528'	
⊙15			4.291'	23.557'	
⊙16			4.296'	23.552'	
⊙17			2.480'	25.368'	BM

Flood Control Of Yabucoa Valley

A. Quiñones T.S.C.O.

FINAL REPORT

FLOOD CONTROL OF YABUCOA VALLEY

PREPARED FOR

COMMONWEALTH OF PUERTO RICO
THE DEPARTMENT OF PUBLIC WORKS

JULY 1969

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS
NEW YORK SAN JUAN

"This report was wholly financed by the Industrial Incentive Funds of the P.R. Industrial Development Company and performed under the supervision of the Department of Public Works of the Commonwealth of Puerto Rico."

Yabucoa - July 1969

CONTENTS

CONTENTS (Cont'd)

LIST OF PLATES

I. SCOPE

This report is submitted in accordance with a contract dated October 31, 1968, between the Department of Public Works, Commonwealth of Puerto Rico, and Tippetts-Abbett-McCarthy-Stratton, Engineers and Architects, New York, N.Y., for a flood control study of the Yabucoa Valley, Puerto Rico.

The report includes the investigations, studies and preparation of preliminary plans for flood protection. Data were obtained from field reconnaissance, from several public agencies and previous reports and studies by others.

The recommended flood protection measures are to be implemented in stages to provide both immediate and long range protection.

All studies, plans and estimates must be considered preliminary, and require more detailed analysis for final design.

II. CONCLUSIONS AND RECOMMENDATIONS

Under present conditions, great floods produce general inundation in the valley, between Routes 906 and 901. Although the flood plain roughness is considerable, the conveyance available in the total section is very large when compared with the conveyance of the channels of the valley. As a result of this, the discharge conveyed by the flood plain is much larger than the discharge conveyed by the rivers, channels and ditches in the valley. The development of present agricultural areas for industrial or housing purposes, will impose severe restrictions in the present conveyance of the valley, since the tolerable frequency of flooding of industrial or housing areas will be much lower than under present conditions. A return period of 100 years was used for study of protection of industrial and housing development areas.

The preliminary development area, proposed for development by Sun Oil Co., will be protected prior to the development of the rest of the valley. Plate 2 shows the proposed system of levees for this stage of the development.

The protection for final industrial development (shown on Plate 5) may be divided in two systems, respectively east and west of the present location of Route 3.

East of Route 3, there would be two major floodways. The South floodway would be confined on the South by the levees already built for the protection of the preliminary development, with the exception of the levee along Cano Santiago which would be relocated to provide for the expansion of the Sun Oil Co. industrial complex. The floodway would be limited on the north side by another levee, to protect the lower areas of the valley.

The North floodway would collect the waters presently conveyed by the Guayanes, Limones and Ingenio Rivers and their tributaries. This floodway would also convey to outflow from the control structures shown on Plate 5 as Bridges I and II.

West of Routes 3 and 905, the system would have a pondage area controlled by the above mentioned bridges. This pondage area would retard and reduce the peak discharges from the upper tributary areas, without inundation of urban areas. This pondage area would be assigned a lower value for development, as it would be flooded with some frequency.

Flood tolerant crops, such as sugar cane, or pasture, would be grown in the area.

The development of the valley would be made in six areas, indicated in Plate 5 with the letters A through F. Area A corresponds to the final development of the Sun Oil Co. facilities. Areas D and E are the urban developments of Yabucoa and of Laura and Martorell, respectively.

The final development protection may be constructed in stages, following the development of the valley. Plate 2 shows an intermediate development plan which could be built to protect the areas of Yabucoa and Ingenio Roig.

It is recommended that new studies be made for further evaluation of the project. The following subjects need a more detailed appraisal which will be conducive to the formulation of the optimum solution within the general outlines of the flood control system.

a. Detailed topographic surveys.

b. Cost analysis of the project.

c. Benefit analysis to be made after details are known about future industrial and housing development of the valley.

d. Study of the possibility of using a multiple purpose reservoir for flood control and water supply, in conjunction with general studies of water resources to be made in the area.

e. Comparison of benefits and costs, and internal rates of return, to appraise the economic feasibility of the sequence of construction.

f. Formulation of an economic model to obtain the optimum solution for the industrial development which would use the flood protection benefits and costs as an input.

g. After the optimum solution is obtained, detailed design of the works recommended for immediate construction should be undertaken. New hydrologic, hydraulic and structural studies may be required.

The preliminary construction cost estimates made for these studies may be summarized as follows:

	Cost	Cost per Acre
Preliminary Protection	1,189,400	1,770
Intermediate Protection	781,300	1,740
Competion of Final Stage of Protection	3,139,100	750

Heavy rainfall on the hills surrounding the valley produces extensive scour. The sediments produced are first deposited on the roads and fields of the valley, and are ultimately conveyed to the Guayanes beach. A sediment control study should be made to prevent, or at least retard, scour. The reservoir proposed by USSCS on the Guayanes River would control about one-third of the tributary area, and its effect on sediment control would only be partial. Sediment control dams should be studied on all the ravines of the tributary area. Sediments reaching the beach will probably require dredging to avoid interference with harbor operations.

Under present conditions, Route 906 is flooded for about 2.5 kilometers during severe floods. It is recommended that relocation studies be made, to ascertain whether it would be convenient to raise the road elevation, or to accept the risk of flooding until the final valley development is effected.

There is no danger of flooding for the Eugenio Maria de Hostos School.

III. INFORMATION AVAILABLE FOR THE STUDY

The following sources of information were used for this study:

US Geological Survey Circular 451, "Floods of September 6, 1960, in Eastern Puerto Rico", 1961.

USGS Quadrangle Maps, Scale 1:20000, Yabucoa and Punta Guayanes and Adjacent Quadrangles.

R.P. Briggs and J.P. Akers; Hydrogeologic Map of Puerto Rico and Adjacent Islands, USGS.

US Soil Conservation Service, Report on Watershed Work Plan, Guayanes River Watershed, Puerto Rico, December 1962.

Information on flood levels obtained by surveys after the flood of 1960 in the Valley of Yabucoa, obtained from the San Juan (P.R.) Office, U.S. Geological Survey.

US Weather Bureau Technical Paper 42, 1961.

T.W. Adair, Memorandum on Guayanes River Dam, Site 1. Engineering and Watershed Planning Unit, USSCS, Fort Worth, Texas.

Information provided by engineers of the Sun Oil Company in San Juan (P.R.) and Philadelphia (Pa.).

Plans of Development provided by the Industrial Development Company, San Juan, P.R.

University of Puerto Rico Agricultural Experimental Station, Soil Map, Scale 1:50000.

Valley cross section surveys provided by the San Juan Office, USSCS.

IV. HYDROLOGY

A. FLOOD ANALYSIS

1. The Flood of 1960. A comprehensive report on the floods of 1960 in Puerto Rico is presented in U.S. Geological Survey Circular 451, "Floods of September 6, 1960, in Eastern Puerto Rico".

Table IV-1 shows a summary of flood discharges presented in the above mentioned report, and the corresponding Myer rating coefficients obtained in cfs per mi. from the equation:

$$C = Q/\sqrt{A}$$

Figure 1 shows the discharges plotted against the drainage areas in sq. mi. at the Myer ratings on log-log paper. The numbers shown beside each point in Figure 1 correspond to those shown in Table IV-1, identifying the drainage areas listed in the above mentioned publication. Some of the Myer ratings of the 1960 flood are among the largest observed in the world.

The San Juan Office of the U.S Geological Survey provided elevations of floodmarks for the 1960 flood as surveyed by official agencies. From this information, the flood boundary and water surface contours were interpolated or estimated, and are shown on Plate 1. The inundation covered most of the valley between Routes 901 and 906 and downstream of Route 3. Plate 1 also shows locations of the cross sections surveyed by the U.S. Soil Conservation Service for its 1962 report.

An estimate of the discharge for the 1960 flood was made by conveyance slope computations, using the flood water-surface contours and the surveyed cross sections. An average water surface slope of 0.00168 was determined for the reach between Route 3 and the dunes at Guayanes Beach. The Manning coefficient for the flood plains was estimated by comparison with retardance coefficients for grassed channels, corrected to take into consideration the sugar cane stalk resistance. These Manning coefficients were estimated at between 0.22 and 0.30. (See Appendix).

The discharge between Sections 8 and 9 was estimated around 70,000 cfs.

... PUERTO RICO FOR

THE FLOOD OF 1960 AND PREVIOUS MAXIMUM DISCHARGES

No.	Stream and place of determination	Drainage Area sq.mi. A	Period of Record	Date	Discharge Cfs	Discharge Cfs per sq. mi.	Myer Rating $C=\frac{Q}{\sqrt{A}}$
1	Rio Grande de Manati at Ciales......	128	1946-60	Sept. 6, 1960	77,300	604	6,830
				Aug. 12, 1956			
2	Rio de la Plata near Cayey...........	42.2		Sept. 6, 1960	51,000	1,210	7,850
3	Rio de la Plata at Proyecto La Plata...	63.1	1959-60	-----do-----	54,500	864	6,850
				Aug. 14, 1960			
4	Rio de la Plata at Comerio Dam.......	140	1914-60	Sept. 6, 1960	101,000	721	8,530
				Sept. 14, 1928	116,000	829	9,780
5	Rio de la Plata at Toa Alta...........	204	1960	Sept. 6, 1960	95,500	468	6,685
				Aug. 14, 1960	9,800	481	686
6	Rio Bayamon near Aguas Buenas......	[1] 9.9		Sept. 6, 1960	[2] 8,000	808	2,540
7	Rio Bayamon Dam near Aguas Buenas...	[1] 18.5		-----do-----	12,800	692	940
8	Rio Turabo near La Plaza............	7.07		-----do-----	[2] 24,000	3,520	9,040
9	Quebrada de las Quebradillas near Caguas.........................	6.25		-----do-----	8,150	1,300	3,250
10	Rio Grande de Loiza at Caguas.......	89.7	1960	-----do-----	71,500	769	7,550
				Aug. 14, 1960			
11	Rio Gurabo near Juncos..............	23.6	1960	Sept. 6, 1960	28,000	1,190	5,700
12	Rio Valenciano near Las Piedras......	6.86	1960	-----do-----	28,800	4,200	11,000
13	Rio Valenciano tributary near Las Piedras.........................	.76	1960	-----do-----	1,770	2,330	2,030
14	Rio Valenciano at Juncos............	15.3	1960	-----do-----	37,100	2,420	9,500
15	Rio Gurabo at Gurabo..............	59.6	1960	-----do-----			
				Aug. 21, 1960	6,600	111	
16	Rio Grande de Loiza at Loiza Dam....	207	1960	Sept. 6, 1960	170,000	822	11,800
17	Rio Grande de Loiza at Carolina......	239	1960	-----do-----	197,000	824	12,750
18	Rio Sabana at Luquillo.............	7.01	1960	-----do-----	8,500	1,210	3,210
19	Rio Fajardo near Fajardo............	14.9	1960	-----do-----	14,500	974	3,760
20	Rio Hicaco near Naguabo............	1.24	1945-54	-----do-----	1,660	1,340	1,490
			1958-60	Oct. 25, 1953	2,560	2,060	2,400
21	Rio Humacao at Las Piedras..........	6.54	1960	Sept. 6, 1960	20,800	3,180	8,120
22	Rio Humacao at Humacao............	10.0	1960	-----do-----	31,600	3,160	8,500
23	Rio Guamani near Guayama...........	5.65	1960	-----do-----	2,580	457	1,090
24	Rio Majada near Salinas............	22.0	1960	-----do-----	11,000	500	2,340
25	Rio Coamo at Coamo...............	48.4	1960	-----do-----	14,300	295	2,050

[1] Drainage area above Cidra Dam not included. [2] Estimated on basis of field survey.

Source: USGS: "Floods of September 6, 1960, in Eastern Puerto Rico Circular 451, 1961."

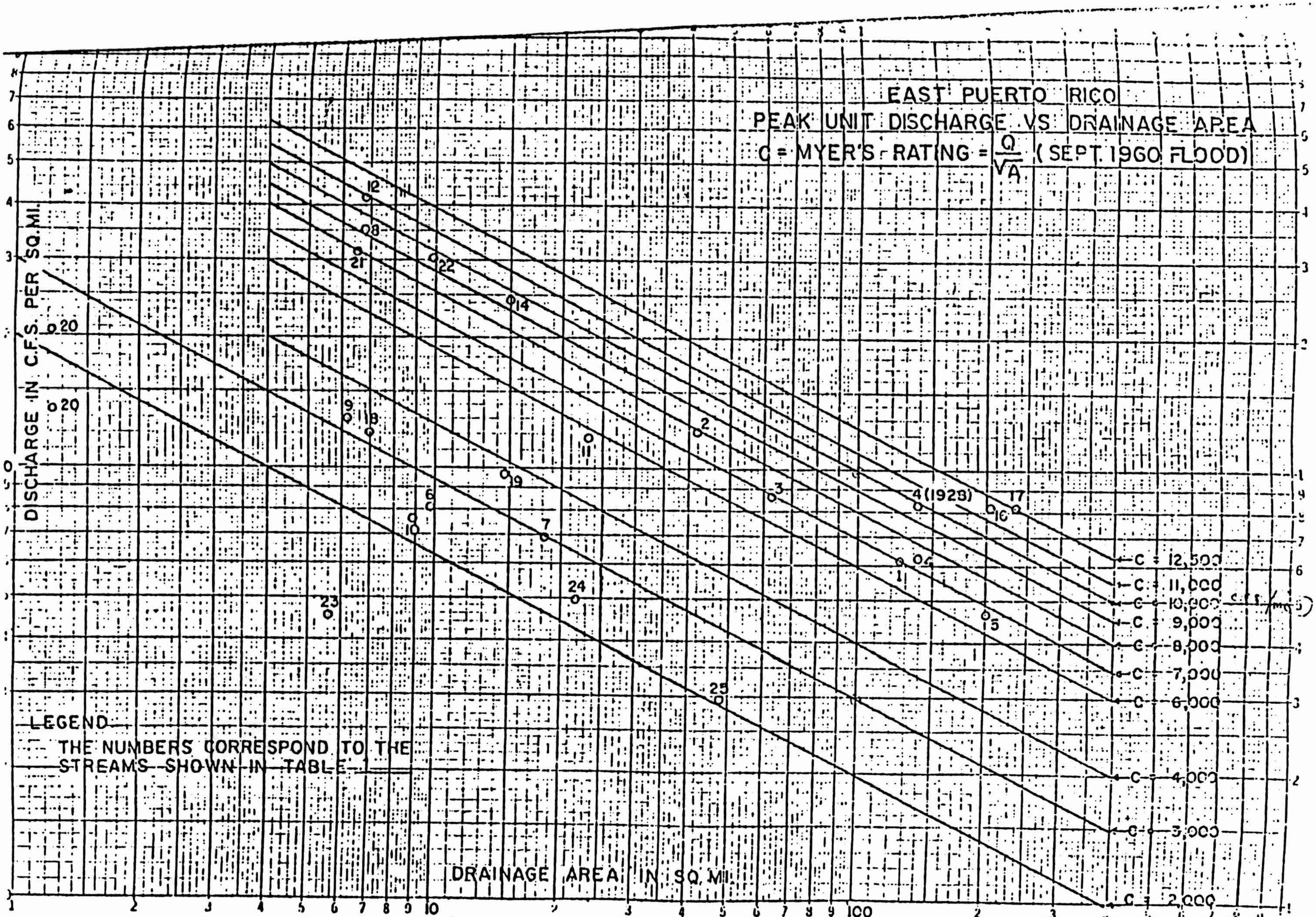

EAST PUERTO RICO
PEAK UNIT DISCHARGE VS DRAINAGE AREA
C = MYER'S RATING = $\frac{Q}{\sqrt{A}}$ (SEPT. 1960 FLOOD)
DISCHARGE IN C.F.S. PER SQ. MI.
DRAINAGE AREA IN SQ. MI.
C = 12,500
C = 11,000
C = 10,000
C = 9,000
C = 8,000
C = 7,000
C = 6,000
C = 4,000
C = 3,000
C = 2,000
4(1928)
LEGEND
THE NUMBERS CORRESPOND TO THE STREAMS SHOWN IN TABLE 1

Table IV-1 and Figure 1 show the Myer ratings computed for other streams of Puerto Rico for the 1960 floods. It may be observed that several ratings were in the order of 10,000.

Consequently, the analysis of the 1960 floods in the Yabucoa Valley was made assuming a Myer rating of 10,000.

2. Flood Frequency Analysis. Continuous discharge records for streams in southeastern Puerto Rico are available for Rio Grande de Loiza at Caguas (drainage area, 89.7 sq. mi.) and Rio Gurabo at Gurabo (drainage area 59.6 sq.mi.) but their periods of record are only 8 years long. Longer flood records are available for other regions of the island and may be transposed statistically to the study area as the flood potentials are similar.

Hence, in addition to the records mentioned above, the 19-year long record for Rio Yahuecas (drainage area, 15.5 sq. mi.) was used for flood frequency analysis.

Use was made of the Ven Te Chow method relating the peak flood discharge (Q) for a given recurrence period, to the mean of the annual series of peak discharges ($\bar{Q}$), the coefficient of variation ($C_v = \sigma/\bar{Q}$), the standard deviation (σ), and a "frequency factor" (K), the latter a function of the recurrence period. The following equation expresses the above relationship.

$$Q = \bar{Q}\,(1 + C_v K)$$

The coefficient of variation may be transposed from basins with long records to the area of interest, because of the general climatological similarity over large areas of the island.

The coefficient of variation for the record of the Rio Grande de Loiza at Caguas was computed to be 0.866.

Computations made with data from Technical Paper 42 by the U. S. Weather Bureau showed that in the area bounded by 18°00' and 18°15'N latitude, and by 66°45' and 66°00'W longitude, the coefficient of variation of the 24-hour annual precipitation is on the average equal to 0.43 or roughly one half the coefficient of variation estimated for the annual floods of the Rio Grande de Loiza. This is in agreement with results of a number of studies on other areas with floods produced by tropical storms.

Consequently, for this study, the coefficient of variation of the annual flood peaks at the Yabucoa Valley was assumed equal to $C_v = 0.86$.

The frequency factors computed with the available records for Yahuecas, Grande de Loiza and Gurabo Rivers are shown in Figure 2. There is a reasonable agreement in the plotted points, and a "K - T Curve" may be drawn for frequency extrapolation. The mean of the annual flood discharge ($\bar{Q}$) was determined for the above three streams, and was related to their drainage areas. The following relationship was obtained graphically:

$$\bar{Q} = 750 A^{0.8}$$

where A is the drainage area in square miles.

Finally, the following expression for Q was obtained by substitution:

$$Q = 750 A^{0.8} (1 + 0.86 K)$$

The return period (recurrence interval) ascribed to Q is that obtained from Figure 2 for the corresponding value of K.

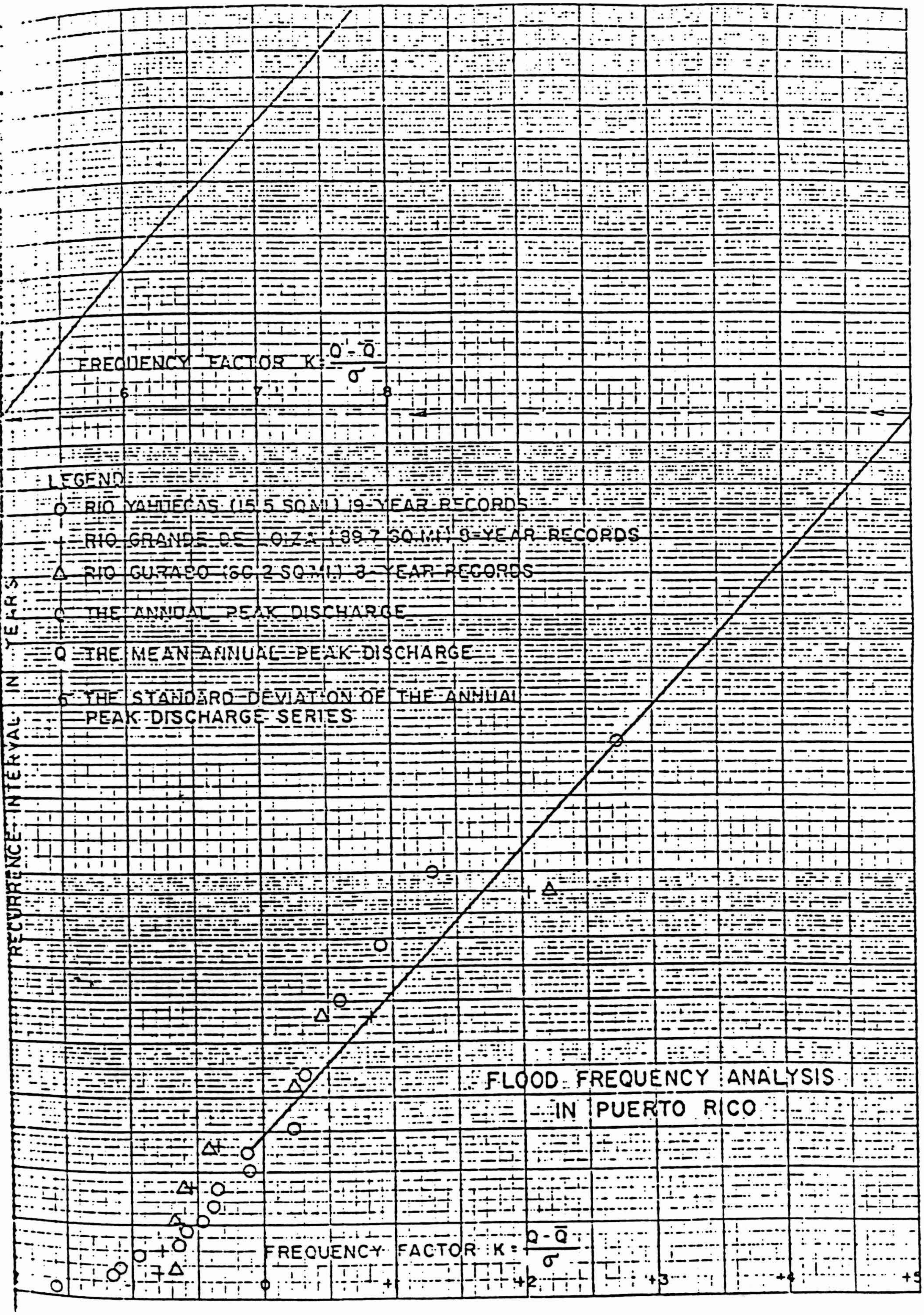
From the above equation, a relationship between the Myer ratings and flood frequencies may be obtained, equating both expressions to the peak discharge:

$$Q = 750 A^{0.8} (1 + 0.86 K) = C \sqrt{A}$$

$$\text{and } C = 750 A^{0.3} (1 + 0.86 K)$$

The above expression for C shows that the Myer rating depends on the area, at least for areas less than 100 sq.mi. considered in this study. The return period ascribed to C is that corresponding to K.

Figure 3 shows a set of curves with the relation between Myer ratings, drainage area and return period. It appears that for a given return period, higher Myer ratings may be obtained for larger areas, due to the greater likehood of obtaining an intense precipitation of relatively small coverage on a larger area than on a smaller one. Figure 3 helps also to explain the large variability of the Myer ratings for the 1960 flood (see Table IV-1. The Myer rating of 10,000 used for the mouth of the Guayanes River (drainage area, 49.4 sq.mi.) corresponds to a return period of about 40 years.



FLOOD FREQUENCY ANALYSIS IN PUERTO RICO
RECURRENCE INTERVAL IN YEARS
FREQUENCY FACTOR K = $\frac{Q - \bar{Q}}{\sigma}$
6
7
8
LEGEND
○ RIO YAHUECAS (15.5 SQ.MI.) 19-YEAR RECORDS
+ RIO GRANDE DE LOIZA (89.7 SQ.MI.) 9-YEAR RECORDS
△ RIO GURABO (60.2 SQ.MI.) 8-YEAR RECORDS
Q THE ANNUAL PEAK DISCHARGE
$\bar{Q}$ THE MEAN ANNUAL PEAK DISCHARGE
σ THE STANDARD DEVIATION OF THE ANNUAL PEAK DISCHARGE SERIES
FREQUENCY FACTOR K = $\frac{Q - \bar{Q}}{\sigma}$
0
+1
+2
+3
+4
+5

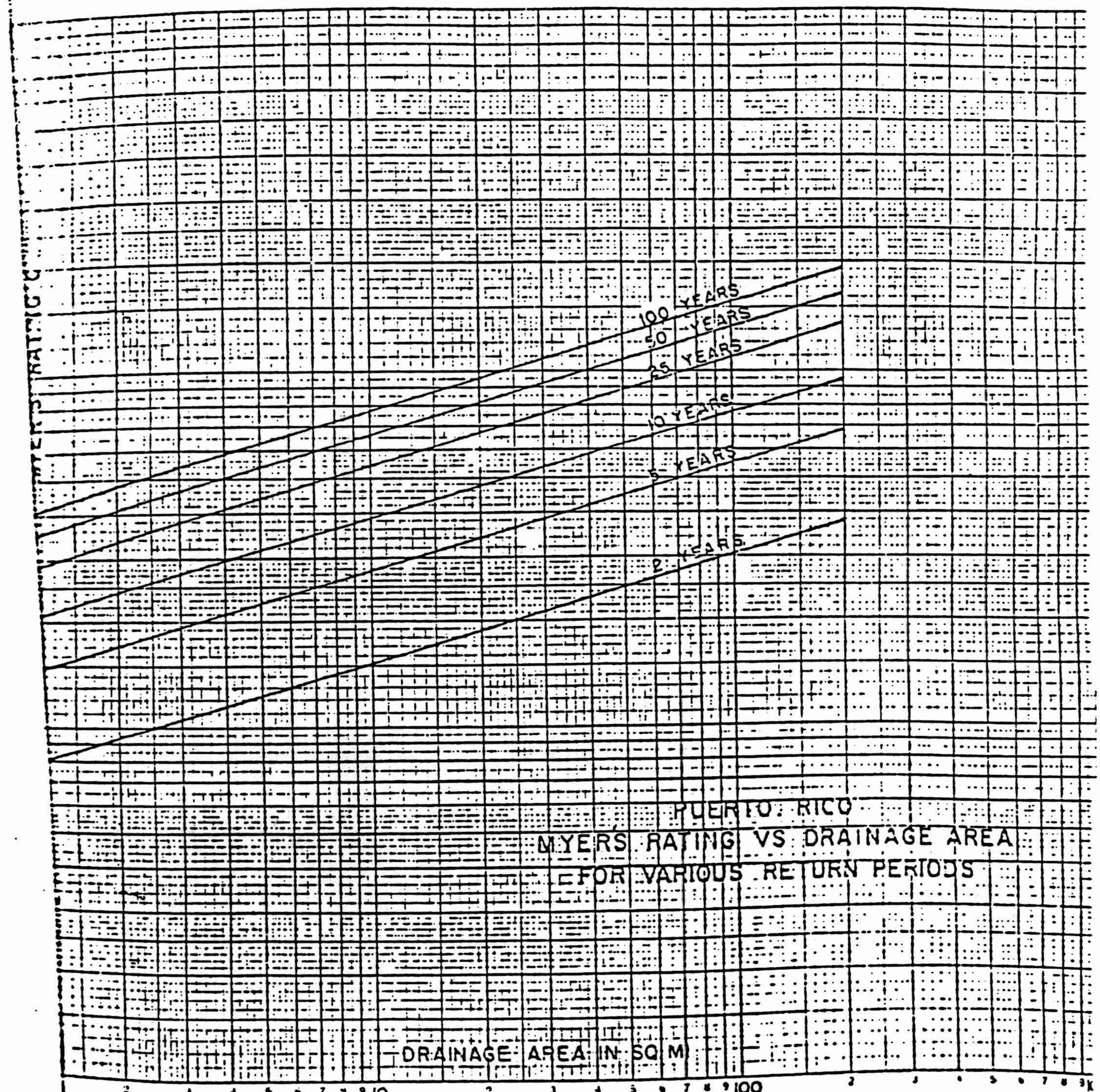

100 YEARS
50 YEARS
25 YEARS
10 YEARS
5 YEARS
2 YEARS
PUERTO RICO
MYERS RATING VS DRAINAGE AREA
FOR VARIOUS RETURN PERIODS
DRAINAGE AREA IN SQ MI
10
100

FIG. 3

USGS Circular 451 states that the 1928 flood was reported higher than the 1960 flood in the Yabucoa Valley which would tend to imply a shorter return period for the 1960 flood.

3. Design Flood Analysis. The crest elevations of the levees protecting housing or industrial developments should be enough to avoid overtopping in the event of a 100-year flood. Table IV-2 shows the drainage areas for the valley cross sections shown in Plate 1 and the corresponding 100-year estimated discharges, according to Figure 3.

These discharges will be used for design of protective facilities for the preliminary industrial development of the valley, to be described later.

TABLE IV-2

DESIGN FLOOD DISCHARGES

PRELIMINARY FLOOD PROTECTION

Section	Drainage Area (A) (sq.mi.)	100-year Myer Rating	$\sqrt{A}$	Discharge (cfs)
1	49.5	12,500	7.03	88,000
5	47.9	12,200	6.91	84,400
6	46.0	12,100	6.78	82,000
7	44.9	12,100	6.69	81,000
8	43.4	12,000	6.58	79,000
9	40.6	11,900	6.36	75,700
10	38.1	11,700	6.17	72,200
11	36.6	11,600	6.05	70,200

The discharges used for design of the flood protection systems for the final industrial development of the valley were determined for three different return periods. A return period of 100 years was considered adequate for design for maximum protection. A return period of 25 years was also considered for comparison of flood control costs and benefits with respect to design for maximum protection. Finally, a return period of 2.33 years, corresponding to the theoretical mean of the annual peak series, appears reasonable for cross section determination of relocated or improved channels within the proposed floodways.

Table IV-3 shows a summary of the discharges considered for the design of the proposed floodways. The sections referred to in Table IV-3 are shown on Plates 5, 7 and 12.

TABLE IV-3

DESIGN FLOOD DISCHARGES

FINAL INDUSTRIAL DEVELOPMENT

Floodway	Station	Section	Drainage Area (sq.mi.)	100-year Flood (cfs)	25-year Flood (cfs)	2.33-year Flood (cfs)
* South	2 + 640	5S	6.49	17,300	12,100	6,100
	0 + 780	7S	5.15	14,400	10,100	5,000
	0 + 220	8S	3.96	11,700	8,200	4,100
	0 + 000	9S	3.05	9,500	6,700	3,300
** North	4 + 660	5N	38.34	71,700	50,200	25,100
	4 + 200	6N	32.74	63,200	44,200	22,100
	2 + 600	9N	32.11	62,200	43,500	21,800
	1 + 500	11N	27.22	55,200	38,600	19,300
	0 + 100	13N	20.66	43,700	30,000	15,300

* See Plates 5 and 7

** See Plates 5 and 12

4. <u>Synthetic Flood Hydrograph for Pondage Analysis</u>. For the analysis of temporary pondage to be obtained upstream of present Routes 3 and 905, which will be discussed later, a synthetic inflow hydrograph was developed assuming the peak of the 100-year flood estimated for the drainage area at the crossing.

The drainage area at the crossing of Routes 3 and 905 is 29.2 sq.mi., about 2.7 times larger than that controlled by the reservoir on the Guayanes River proposed by USSCS.

A synthetic unit hydrograph was obtained using Snyder's method. From analysis of the 1960 flood hydrograph for the Rio Grande de Loiza at Caguas reported in USGS Circular 451, the Snyder's coefficients were estimated at $C_t = 1.0$ and $640 C_p = 750$. A comparison with unit hydrographs derived for similar basins in Panama indicated that these coefficients are reasonable. From the USGS 1:20,000 map, the lenght of the longest tributary was estimated to be 13.3 mi. and the length from the center of gravity of the basin to the crossing was estimated to be 6.55 mi. Assuming a 6-hour unit rain, the lag time t_p was calculated to be about 5 hours and the peak of the unit hydrograph at 4230 cfs.

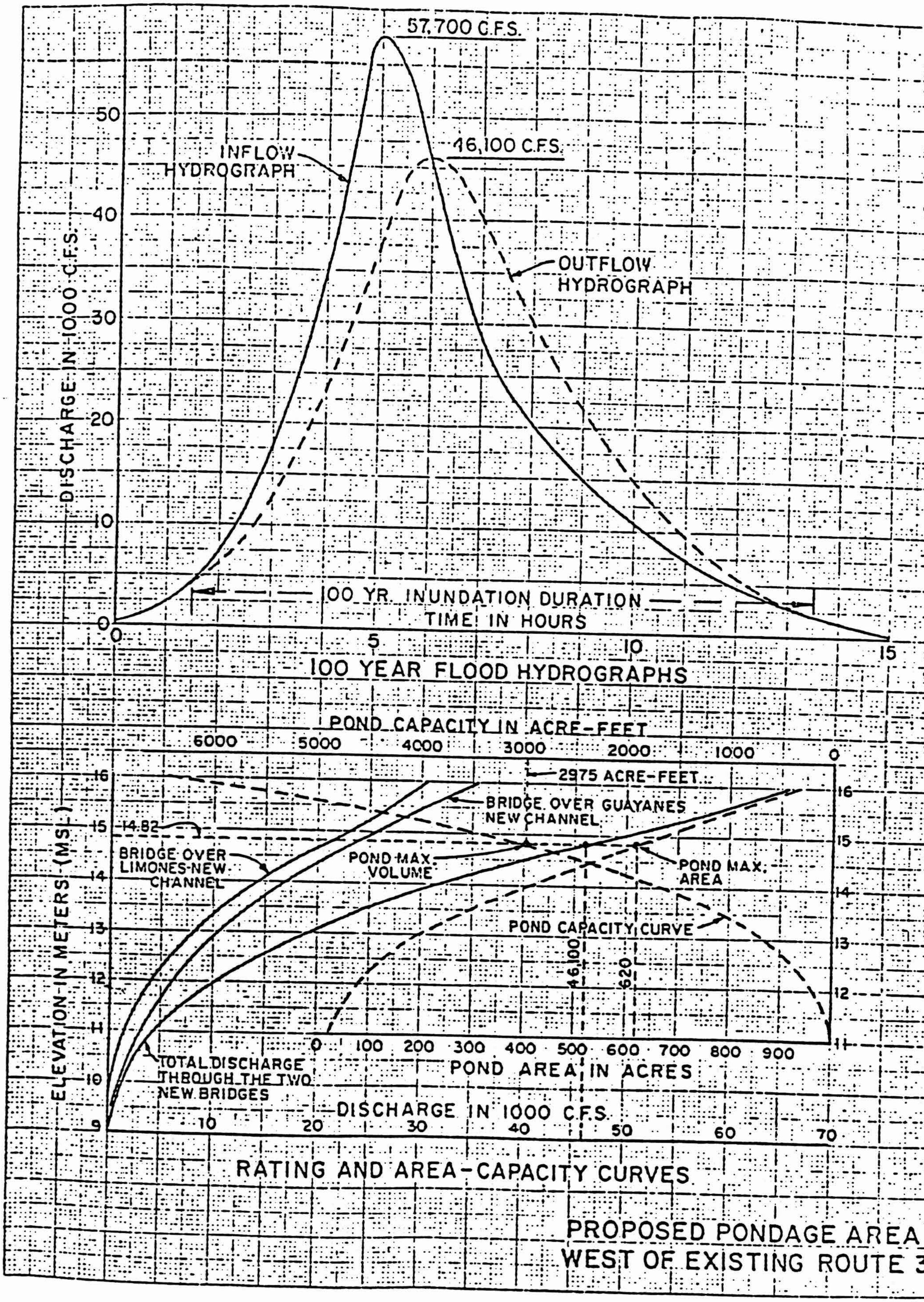
From the frequency analysis reported above, it was concluded that the peak discharge for the 100-year flood over the area tributary to the crossing of Routes 3 and 905, would be 57,700 cfs. Comparing this discharge with the peak of the unit hydrograph, it appears that the direct runoff for the 100-year flood should be greater than 13 in.

US Weather Bureau Technical Paper No.42 reports a depth of 10 in. for the 6-hour, 100-year precipitation in the area, which is less than the above estimate.

However, the analysis of the 1960 storm reveals higher depths for six-hour durations. USGS Circular 451 reports that during the storm of 1960, the runoff measured at Caguas was 10.1 in. for a drainage area of 89.7 sq. mi. The precipitation recorded on September 6, 1960, at 8:00 hours at Caguas was 11.4 in. Apparently, the precipitation began around 00:00 hours, since the Caguas River had a discharge of only 200 cfs at that time. Circular 451 also states that the high intensity precipitation lasted from 21:00 hours of September 5, until 3:00 or 4:00 hours of September 6. It appears reasonable to assume that the Caguas precipitation ocurred mostly within 4 hours. During the same storm, 14.6 in. of rainfall were recorded at Naguabo, some 25 mi. NNE of Yabucoa. It appears that this precipitation occured in about 7 hours.

It is concluded that a direct runoff of about 13.4 in. is reasonable for the 100-year storm and the duration of 6 hours. The inflow hydrograph shown in Fig. 4 is obtained multiplying the ordinates of the unit hydrograph by 13.44 in. and adding a base flow of 200 cfs., which results in a peak of 57,700 cfs.

The inflow hydrograph was routed through the pondage area just west of Routes 3 and 905 to be discussed later. For this, it was assumed that two new bridges would be built over the relocated channels of the Limones and Guayanes Rivers with spans of 40 and 60 meters, respectively. The bottom elevations of the two channels at the crossings, would



PROPOSED PONDAGE AREA
WEST OF EXISTING ROUTE 3

FIG. 4

be 9.5 and 9.0 meters, respectively.

Figure 4 shows the rating curves of these two control structures, and the area - capacity curves of the pond.

The peak outflow for the 100-year flood was computed to be 46,100 cfs, with a maximum elevation of 14.8 m, a maximum storage volume of 2975 acre-feet and a maximum pond area of 620 acres. The duration of the inundation in the ponding area was estimated at 12 hours. This duration would not produce substantial damage to sugar cane or pasture.

Further analysis is required to obtain the best combination of levee heights, bridge spans, and extent and duration of inundation.

The peak outflow of 46,100 cfs would correspond to the 100-year flood from an uncontrolled tributary area of about 22 sq.mi., instead of the 29.1 sq. mi. tributary to the pond. In other words, the pond would have an effect equal to a reduction in the tributary area of 7.1 sq.mi.

This reduction was applied to the areas tributary to sections located downstream on the North Floodway, to be discussed later, for computation of the corresponding 100-year flood peaks. The following Table IV-4 shows the estimated 100-year flood peaks for the North Floodway.

TABLE IV-4

DESIGN FLOOD DISCHARGES

FINAL INDUSTRIAL DEVELOPMENT

North Floodway

Station	Section	Drainage Area (sq.mi.) Actual	Drainage Area (sq.mi.) Reduced	100-year Flood (cfs)
4 + 660	5N	38.34	31.2	60,800
4 + 200	6N	32.74	25.6	51 900
2 + 600	9N	32.11	25.0	50,900
1 + 500	11N	Peak outflow from bridges I & II		46,100
0 + 100	13N	Peak outflow from bridge I		24,000

Table IV-4 shows reduction only for sections of the floodway receiving flow from the whole tributary area upstream of Route 3. Sections

1 + 500 and 0 + 100 receive flow only from the Limones and Guayanes areas. Comparing the discharges reported in Tables IV-3 and IV-4, it may be observed that the proposed pond would have a substantial effect in reducing the discharges of the floodway.

V. SOILS AND FOUNDATION CONDITIONS

A. GENERAL

The soils and foundation conditions were determined from 1:50,000 scale soil map prepared by the University of Puerto Rico Agricultural Experimental Station, a memorandum on the Guayanes River Dam, site 1, prepared by T.W. Adair, Engineering and Watershed Planning Unit, SCS, Fort Worth, Texas, and by field reconnaissance. The field reconnaissance consisted of spot checking of the surface deposits along the proposed levees. The information presented in this report is preliminary and should be confirmed by subsurface exploration prior to the design stage.

B. LEVEES

1. Foundation Conditions. The foundation materials for the proposed levees consist of deep alluvial deposits varying from clayey, sandy silt to plastic clay. Clean sand is deposited along the sea shore and along the Guayanes River to a distance of about 1/2 mile from the mouth of the river. Silty, clayey sand deposits generally occur along the present course of the Guayanes River and the Cano de Santiago. The sandy, clayey silt surface deposits of the flood-plain are underlain by plastic clay. Locally, the clay is at ground surface. No thick organic silt or soft clay deposits unsuitable to support the shallow embankment of the proposed levees are believed to exist in the project area. The topsoil is about 2 ft. thick and should be removed from the foundation prior to construction of the levees.

2. Construction Materials. Alluvial deposits in the project area located above the ground water table and free of high organic content are suitable for levee construction. The silty, clayey sands of the existing levees and the silty, clayey sand deposits along the present course of the Guayanes River and the Cano de Santiago are probably the best sources of construction materials. If materials from the existing levees are used, it will be necessary to undertake extensive clearing and stripping to remove existing vegetation.

C. EARTH DAM

1. Foundation Conditions. At the flood-plain the foundation materials consist of 60 to 80 feet of alluvium underlain by weathered granodiorite. The alluvial materials consist of a surface layer of 15 to 20 feet of soft clayey silt underlain by silt, clayey silt, silty sand and clayey sand

materials. Bedrock at the site consists mainly of granodiorite, diorite and gabbro. The rock appears at the surface at both abutments, but is deeply weathered. There is an inactive fault which passes near the south end of the dam. The proposed spillway will be supported on an eath foundation.

2. <u>Construction Materials.</u> Sufficient amounts of silty, clayey sand materials are available from nearby flood-plain deposits for the construction of the dam. The natural moisture content of these materials may be high. This condition should be considered in the design of the embankment dam.

VI. FLOOD PROTECTION SCHEMES

A. GENERAL

The flood protection development should follow the stages of housing, industrial and agricultural development in the valley. Within the scope of this report, three different stages of protection have been contemplated as follows:

1. Preliminary flood protection (Sun Oil Co. Industrial Facilities)
2. Intermediate flood protection .
3. Final industrial development and related protection.

B. PRELIMINARY FLOOD PROTECTION

1. Scheme Description. Plate 2 shows the areas to be occupied by the Sun Oil Company, for their proposed industrial complex, tank farm and harbor facilities. It may be observed that most of these areas were flooded during the 1960 flood, and should therefore be protected against similar events. This protection was considered the preliminary stage of the general valley protection.

New levees are proposed along the right bank of Cano Santiago to protect the tank farm and the harbor area. The existing levees at the area proposed for the industrial complex have crest elevations adequate for protection against flooding during the 100-year event, but their alignment may not be suitable for the layout of the industrial area.

A rectification of the Cano Santiago has been proposed in the area of the harbor facilities. It is recommended that the relocated chanel be made 60 meters wide, with stable side slopes.

2. Backwater Elevations. To obtain water surface elevations for design of the levees around the industrial comples, tank farm, and

harbor area, backwater computations were made using the TAMS 1130 computer program. Fig. 3 shows the elevation of a floodmark and its agreement with the computed backwater curve for the 1960 flood.

A series of computations was then made for preliminary protection conditions, assuming that the new Route 3 layout will include a trestle for the section between the North boundary of the flood plain and the harbor area. It is very necessary that the route not impose further constriction in the valley conveyance. Bridges in the Yabucoa Valley tend to clog with debris, and during floods their waterway section becomes substantially reduced. It was assumed that the backwater produced by the trestle will be negligible.

Table VI-1 shows the computed water surface elevations for the 1960 flood and for the 100-year flood.

The constriction imposed at the mouth of the valley by the harbor and tank farm developments will increase the water-surface elevation there with respect to present conditions and for the 1960 flood, by no more than 0.50 meter. This increase will gradually diminish toward the upstream end of the development, and will become negligible at the present location of Route 3.

The low-steel elevation at the present Route 3 bridge over the Cano Santiago is 13.2 m MSL. The water surface elevation computed for Station 11 would be the tailwater for this bridge. The proposed preliminary protection would not materially change the water surface elevation with respect to present elevations from the bridge to Station 8. The bridge would have almost 1.3 m of freeboard above the 1960 flood.

From the information given in Table VI-1, it may be concluded that the proposed preliminary valley development would not materially increase the present danger of flooding of the city of Yabucoa or the Roig Sugar Mill. However, protection should be provided there, to avoid recurrence of the inundation experienced in past major floods. This protection will be obtained after the intermediate or final protection plans are developed, as discussed below.

TABLE VI-1

WATER SURFACE ELEVATIONS FOR PRESENT CONDITIONS AND PRELIMINARY FLOOD PROTECTION

(mMSL)

USSCS Section	Present Conditions, 1960 Flood	Preliminary Flood Protection 1960 Flood	Preliminary Flood Protection 100-yr Flood
1*		1.4	1.8
2*		2.5	2.9
3*		2.7	3.1
4	3.0	3.3	3.7
5	3.8	4.1	4.6
6	4.3	4.8	5.2
7	4.7	5.1	5.5
8	6.0	6.1	6.6
9	8.3	8.1	8.4
10	10.5	10.5	10.7
11	11.8	11.9	12.1
12			14.4
13			16.0
14			16.8
15			18.7

* Adapted from USSCS topographic information.

A freeboard of 0.50 meter has been added in determining leeve crest elevations.

The levees on the Laja Creek will follow the alignment shown on Plate2, which was agreed upon at a meeting with representatives of the various agencies interested in this project. The cost for the levees along this creek has been considered separately in the cost estimates.

C. FLOOD PROTECTION FOR FINAL VALLEY DEVELOPMENT.

1. General. Flood protection for final valley development may be achieved through the following measures:

a. Flood peak reduction through flood control reservoirs.

b. Flood peak reduction through pondage areas upstream of bridge crossings.

c. Floodways

d. A combination of the above measures.

The present levees along the banks of the river channels are only effective in preventing minor floods from inundating the valley, but probably contribute to increased flooding during large floods, because they are too close together and obstruct the valley conveyance. This distance between the levees must be commensurate with large flood discharges.

2. Design Flood Frequencies. Flood protection of housing and industrial facilities requires relatively rare design floods, as the losses and hazards due to flooding may be considerable. As stated above, a return period of 100 years was used for protection of housing and industrial areas. This period was also used by the U.S. Soil Conservation Service for their studies in 1962. In addition, discharges and flow lines were determined for a flood with a 25-year return period for later economic comparison of costs and benefits with those for the 100-year design flood.

For protection of agricultural areas, the U.S.Soil Conservation Service considered channel capacities required to remove runoff from a 24-

hour storm with a three year return period. If diverted flood-waters from adjacent drainage areas were conveyed through these canals, added capacity would be provided to assure the removal of the diverted waters from a 24-hour storm with a five-year return period.

The industrial and housing developments of the valley will create more critical flood conditions. Protection of developed areas may proceed by stages according to the development of the valley, using a return period of inundation of 100 years for each protected area. Higher frequencies of flooding may be used for the areas devoted to floodway and retardation pond.

The areas to be devoted to ponding and floodways are important, in view of the large flood discharges to be expected in the valley. Although subject to relatively frequent flooding, these areas have an economic value if devoted to pasture or sugar cane, which are flood tolerant. A low-flow channel was designed for each floodway, to prevent continuous flooding of the area between levees.

It is proposed that floodways be free of encroachments except for the allowed crops, so that the floodwaters will recede rapidly after the passage of the flood and damage to the crops will be avoided.

3. <u>Analysis of Flood Reduction through Reservoirs or Pondage.</u> From the site investigations made by the U.S. Soil Conservation Service, it was concluded that only the Guayanes River valley provides sites suitable for flood control reservoirs. This was confirmed by field inspection.

The most downstream site on the Guayanes is located less than one kilometer upstream of the confluence with Guayabo Creek. The drainage area at this site is 28.2 sq.km. or 10.9 sq.mi. (see Plate 5).

The total drainage area of the Guayanes River and its tributaries at the mouth at the Caribbean Sea is 49.5 sq.mi., so that the drainage area controlled by a reservoir located as proposed by the USSCS is about 22 percent of the total area tributary to the river. The remainder is 78 percent of the tributary area or 38.6 sq.mi.

From the flood frequency analysis discussed above, it is concluded that the reservoir would at most reduce the 100-year peak discharge at the mouth of the valley from 88,000 cfs to about 72,000 cfs. Therefore, the peak reduction would be only about 18 percent of the peak under present conditions. To obtain this peak reduction, the live storage capacity of the reservoir must be equal to the 100-year flood runoff, which was estimated to be 13.4 in. over the contributing area of 10.9 sq. mi. This is more than double the capacity proposed in the USSCS plan, namely 3555 acre-feet, or about 6 in. over the contributing area. Consequently, the USSCS reservoir would produce a reduction of the peak at the mouth of the valley, equivalent to a reduction of the tributary area of about 5 sq. mi.

Another procedure to obtain a reduction of the peak discharge in the valley would be to pass the Guayanes flood through a pond located upstream of present Routes 3 and 905. With such a plan, the bridges of Route 3 would act as control structures.

A preliminary flood routing analysis, described in Chapter IV, was made to estimate the pondage efficiency. It was concluded that a pond of 620 acres in area and 2975 acre-feet total capacity could reduce the 100-year flood peak in the same amount as if the valley tributary area was reduced by 7.1 sq.mi.

The decision whether to build a dam on the Guayanes should be reached only after an analysis of the alternative costs and benefits involved. Such analysis should consider the possibility that the land to be occupied by the pond will be required for urban or industrial development. The construction cost of the pondage solution will be considerably lower than that of the flood control reservoir. However, the land allocated to pondage must be restricted in use due to the relatively frequent flooding to be expected. Some housing relocation may be necessary for this solution. In the final industrial and housing development of the valley, the reservoir structure may become mandatory. However, its construction could be postponed advantageously during the first years of the valley development.

4. Scheme Description. The proposed scheme of flood protection for final industrial development contemplates the combined use of floodways and pondage at the crossing of the present Route 3. Plate 5 shows a general map of the valley, its tributary areas, and the areas of flood protection.

The valley would be divided into sub-areas by levees as shown on Plate 5 and indicated in Table VI-2.

The area of protection was measured within the limits of the 1960 flood shown on Plate 1. The total protected area in the valley is the sum of areas A, B, C, D, E, and F or 5,295 acres.

The proposed floodways are shown on Plate 5. East of the present location of Route 3 there would be two major floodways formed by the levees limiting the protected areas. Both would have low-flow channels.

TABLE VI-2

PROTECTED AREAS - FINAL INDUSTRIAL DEVELOPMENT

Area (acres)		Designation
	670	Preliminary development
A.	1440	Industrial development, including the preliminary development.
B.	210	Industrial develoment
C.	435	Industrial development
D.	190	Yabucoa
E.	500	Martorell and Laura
F.	2520	Agricultural or industrial development, including Roig Sugar Mill.

The south floodway would follow essentially the Cano Santiago except that there would be a relocation necessary for the contemplated final development of the Sunoco industrial complex. (Area A). The relocated channel would have a bottom width of 20 meters and side slopes of 1 vertical on 2 horizontal.

The levee built for the preliminary development on the south side of the present Cano Santiago would be relocated.

Plate 5 shows the general alignment of the South floodway. Plates 6 and 7 show cross sections and a longitudinal profile for the floodway from a station (0 + 00) just downstream of the existing Route 3 bridge to the proposed harbor facilities (Station 5 + 020).

The width of the floodway would vary between 60 m and 100 m. and the levee height would vary between 1.0 and 5.3 m above the ground elevation. The present levees along Cano Santiago would be removed or breached so that they would not form an obstruction to the improved waterway.

The maximum discharge through this floodway, for the 100-year flood would vary between 9,500 cfs at Station 0 + 000 and 17,300 cfs at Station 5 + 020.

The crossing of the proposed new Route 3 would be by a section of the trestle built during the preliminary development, which would not constrict the waterway. Within the scope of this report, it has been assumed that the head loss imposed by this structure is small and that the freeboard allowance will be adequate for the 100-year flood.

The crossing of present Route 182 would be through the existing bridge over Cano Santiago, which would be left in place and limit the discharge to channel capacity.

The North floodway would convey the flow presently carried by the Ingenio, Limones and Guayanes Rivers and their tributaries. I would have its mouth at the present outlet of the Guayanes River. The width between levees would be between 300 and 600 m. The levee height above the natural ground would vary between 1.0 and 3.9 m. Existing levees along the banks of the river would be removed or breached at several sites so that they would not present obstructions to the improved waterway.

Plates 8 through 11 show the cross sections of the north floodway. Plate 12 shows the longitudinal profile.

The proposed pond area would be located just west of Routes 3 and 905. Two control structures, shown in Plate 5 as bridges I and II, would be located, respectively, on the relocated channel of the Guayanes River and 1.6 km to the North. Plate 13 shows the proposed preliminary layout.

Bridge I (Plate 5) would have a span of 60 m. The channel bottom would be paved with a concrete slab at elevation 9.0 m, and suitable riprap protection would also be provided at the banks and bottom.

Bridge II (Plate 5) would have a span of 40 m and the bottom slab would be at elevation 9.5. Rip-rap protection would be provided, as for Bridge I.

Levees would be built along the west side of Routes 3 and 905, to elevations adequate to pass the 100-year flood. Within the area of the pond, this elevation would be 15.5 m. Plate 14 shows the layout proposed for these levees.

West of Routes 3 and 905 the scheme contemplates a system of levees shown on Plate 5, which would confine the streams to their channels and avoid flooding of urban areas. The required levees would not be higher than 2.5 m above the ground.

5. Intermediate Protection Plan. Discussions were held with the Commonwealth agencies interested in this project regarding the areas to be protected after the preliminary protection plan is completed.

From these discussions, the consultants concluded that the second step toward the general valley protection should be to protect the low-lying areas of Yabucoa City and the sugar mill. The planned industrial development on the northern areas of the valley would be protected afterward. Plate 2, shows the proposed intermediate protection which will accomplish this purpose and will become a part of the final protection scheme. Essentially, this system would be formed by a levee and a diversion of the Guayanes River to the new Bridge I, having a span of 60 meters, as described above. The downstream end of the levee would be located just downstream of Section 10.

From the backwater computations made for conditions of the preliminary development (Plate 4), the tailwater elevation at the end of the levee is estimated at about 10.0 m MSL. This water surface elevation would not flood the sugar mill or the low lying areas of Yabucoa, according to the USGS 1:20,000 quadrangle map used as a basis for these studies.

The levee would also protect Yabucoa on the north and east, as shown on Plate 2. The system would include a diversion of Quebrada Aguas Claras to avoid flooding of the Yabucoa area enclosed within the levee.

The Cano Santiago would remain in its present channel and would pass through the existing bridge of Route 182 as for final development. The bridge capacity would limit the discharges to the capacity of the channel within the present levees. Consequently, the city would not be flooded.

6. Ditches, Diversions and Other Secondary Works. Plate 5 shows the diversion channels required to relocate the major streams of the valley, as well as other secondary ditches that may be needed for drainage of protected areas.

The Plate also shows short diversion embankments that will be required to direct tributary streams into the floodways. The most important embankment will be required to divert the Guayanes River into the proposed relocated channel. Rip-rap protection will be required for these diversion embankments.

TABLE VI-3

COST ESTIMATE

STAGE	QUANTITIES AND COSTS IN THOUSAND CUBIC METERS OR DOLLARS									TOTAL COST DOLLARS
	STRIPPING at $0.75 per cu. m.			EXCAVATION at $1.00 per cu. m.				OVERHAUL $0.50/cu. m.	FILL $1.50/cu. m.	
	Channel	Levee	Borrow	Channel	Levee	Borrow	Dredging			
1. PRELIMINARY DEVELOPMENT										1,189,400
1. Tank farm and harbor protection										938,000
a. Quantity	38.4	32.9	54.1	34.7		146.9	271.7	146.9	154.4	
b. Cost	28.8	24.7	40.6	34.7		146.9	271.7	73.5	231.6	852,500
c. Engineering & Contingencies – 10%	2.9	2.5	4.1	3.5		14.7	27.2	7.4	23.2	85,500
2. Industrial Development										66,000
a. Quantity		5.1	5.6			18.7		18.7	15.9	
b. Cost		3.8	4.2			18.7		9.4	23.9	60,000
c. Engineering & Contingencies – 10%		.4	.4			1.9		.9	2.4	6,000
3. Laja Creek Diversion										67,600
a. Quantity	8.6	9.2		19.3					16.1	
b. Cost	6.5	6.9		19.3					24.2	56,900
c. Engineering & Contingencies – 10%	0.7	0.7		1.9					2.4	5,700
d. Culvert										5,000
4. Camino Nuevo Creek Diversion										117,800
a. Quantity	14.7	10.6		33.1					27.6	
b. Cost	11.0	8.0		33.1					41.4	93,500
c. Engineering & Contingencies – 10%	1.1	0.8		3.3					4.1	9,300
d. Culvert										15,000
2. INTERMEDIATE DEVELOPMENT										781,300
1. Guayanes New Channel and Right Bank Levee (Initial Stage)										662,100
a. Quantity	59.0	30.0	19.4	73.0		64.8		64.8	117.1	
b. Cost	44.3	22.5	14.6	73.0		64.8		32.4	175.7	427,300
c. Engineering & Contingencies – 10%	4.4	2.3	1.5	7.3		6.5		3.2	17.6	42,800
d. Bridge 1										192,000
2. Aguas Claras Diversion										119,200
a. Quantity	14.8	17.2		33.4					27.9	
b. Cost	11.1	12.9		33.4					41.9	99,300
c. Engineering & Contingencies – 10%	1.1	1.3		3.3					4.2	9,900
d. Culvert										10,000
3. COMPLETION OF FINAL DEVELOPMENT										3,139,100
a. Quantity	99.2	196.7		839.6	276.8				893.8	
b. Cost	74.4	147.6		839.6	276.8				1,340.7	2,679,100
c. Engineering & Contingencies – 10%	7.4	14.8		84.0	27.7				134.1	268,000
d. Bridge 2										192,000
PRELIMINARY, INTERMEDIATE AND FINAL DEVELOPMENT										5,109,800

7. Preliminary Cost Estimates. Preliminary quantity estimates were made to obtain a reasonable estimate of costs of construction for every stage of the proposed plan. Table VI-3 presents a summary of quantities and construction costs for the proposed stages of development. Land costs have not been included.

The costs per acre of protected area of the several stages of protection are given in Table VI-4.

TABLE VI-4

CONSTRUCTION COSTS PER UNIT AREA

Stage	Construction Cost Dollars	Protected Area Acres	Cost per Acre Dollars/Acre
Preliminary	1,189,400	670	1,770
Intermediate	781,300	450	1,740
Completion of Final Stage *	3,139,100	4,175	750
Final Stage of Development **	4,105,800	5,295	780

* Areas A, B, C, D, E, F less Preliminary and Intermediate: 5295 - 1120 = 4,175 Acres.

** Areas A, B, C, D, E, and F, including area of Preliminary Development. (If built concurrently).

The construction cost per acre of preliminary and intermediate protection stages are considerably higher than the cost to complete the final stage of protection, due to the relatively small, but valuable lands in the first two stages. Table VI-4 gives the construction costs without discounting them to present value, because there are no plans as yet for contruction starts of the final development works.

For reference, Table VI-4 gives the cost per acre of protected land obtained if the final protection development is undertaken concurrently. This cost includes the diversion of Laja and Camino Nuevo Creeks, which was included in the Preliminary Stage of Protection, but excludes work of the Preliminary Stage that would not be required in this case.

APPENDIX

Determination of the Manning's Coefficient for the Yabucoa Valley.

The Manning's coefficient was determined by studying the elevations of the marks left by the 1960 flood and interpolating water surface contours. Cross sections of the valley were provided by the San Juan Office of the U.S. Soils Conservation Service. The locations of several of these sections are shown on Plate 1.

Combining water surface elevation and cross sectional data, factors for uniform flow conditions were determined for Sections 8 and 9. These factors are defined as: $AR^{2/3}$, where A is the cross sectional area and R is the hydraulic radius. Sections 8 and 9 were selected because they are representative of the valley conditions, and also because of their vicinity to the Sunoco industrial complex. The average value of $AR^{2/3}$ was estimated at 8940 $m^{8/3}$, for the flood plain, and 450 $m^{8/3}$ for the channel sections.

Calling n_1 the value of Manning's n for the channels, and n_2, that for the flood plain, the flood discharge may be expressed as:

$$Q = (1/n_1)\ 8940\sqrt{s} \quad + \quad (1/n_2)\ 450\sqrt{s}$$

The surface slope, s, of the 1960 flood was estimated at 0.00168. Consequently:

$$Q = (366.45/n_1) + (18.45/n_2)$$

The n value for rivers may be estimated at 0.030 to 0.035. The discharge for the 1960 flood may be estimated from the relationship given in Chapter IV of the main report.

Table A-1 shows the estimates for n_2, assuming that the 1960 event had recurrence periods of 100 and 50 years, respectively.

It is observed that the roughness coefficient for the valley over bank section is very high. It is probably higher than the value adopted by the USSCS for their studies (n = 0.2).

The values given by Ven Te Chow (Open Channel Hydraulics, McGraw Hill, 1959) for grassed channels may be applied by analogy to the flood plain in the Yabucoa Valley. These values are given in terms of a factor VR, where V is the average velocity and R is the hydraulic radius. For

TABLE A-1

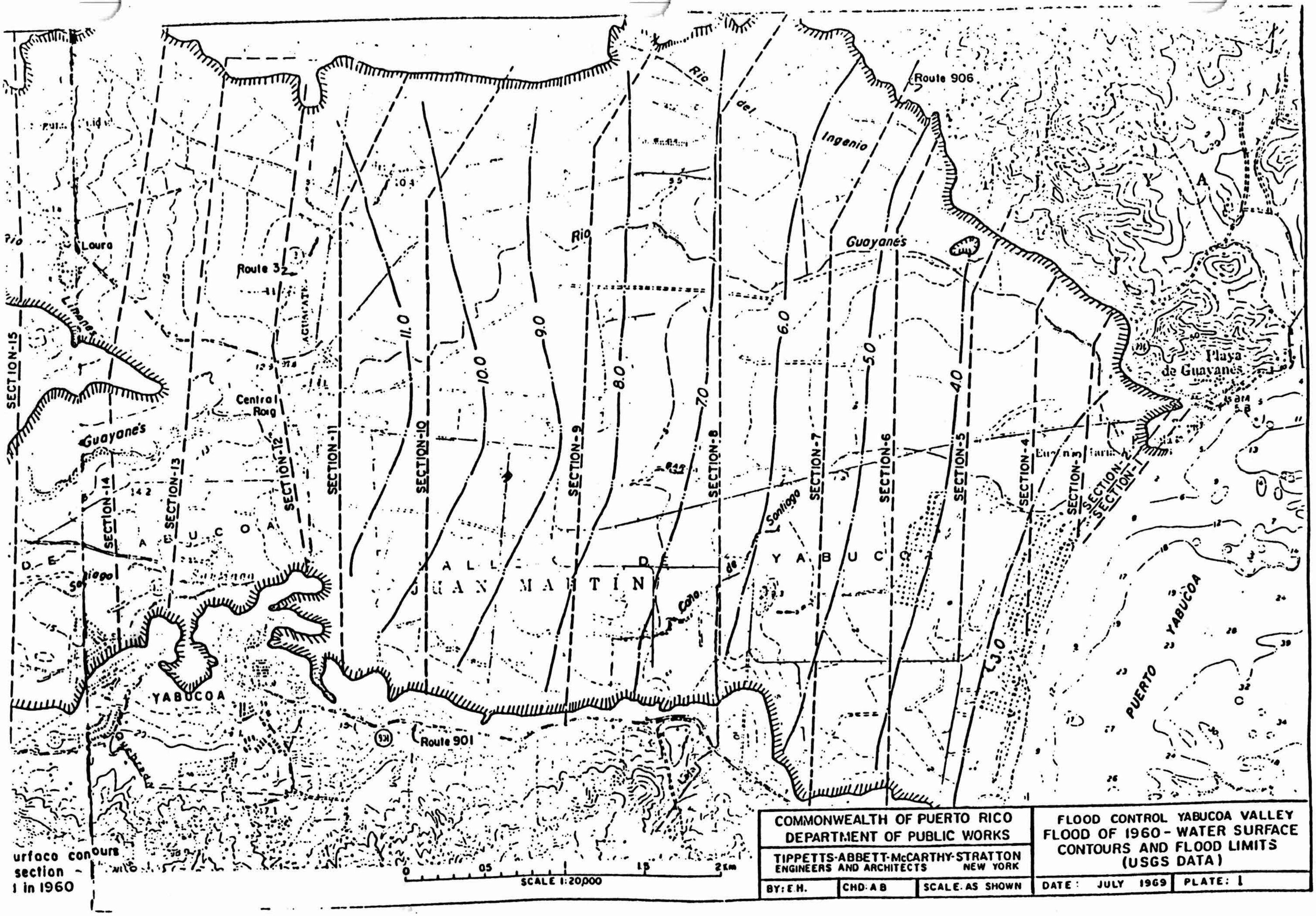
Estimates of Manning's n Values for Yabucoa Valley

Flood of 1960

Assumed Return Period yrs.	Discharge cfs	n_2	n_1	n_1/n_2
50	65,700	0.030	0.233	7.78
50	65,700	0.035	0.221	6.31
100	77,100	0.030	0.294	9.81
100	77,100	0.035	0.275	7.85

sections 8 and 9, VR may be estimated at 4.6. Sugar cane is much taller than grass, so some allowance should be made when using Ven Te Chow's coefficients. Without such allowance, the Manning's coefficient estimated with Chow's information is of the order of 0.15. The values given in Table A-1 are higher, but may be considered reasonable in view of the large degree of obstruction presented by the sugar cane. In addition, an allowance must be made for section obstructions and ponding due to valley microrelief.

It is concluded that n_1 should be estimated at between 0.22 and 0.30. For the backwater computations it was assumed equal to 0.25.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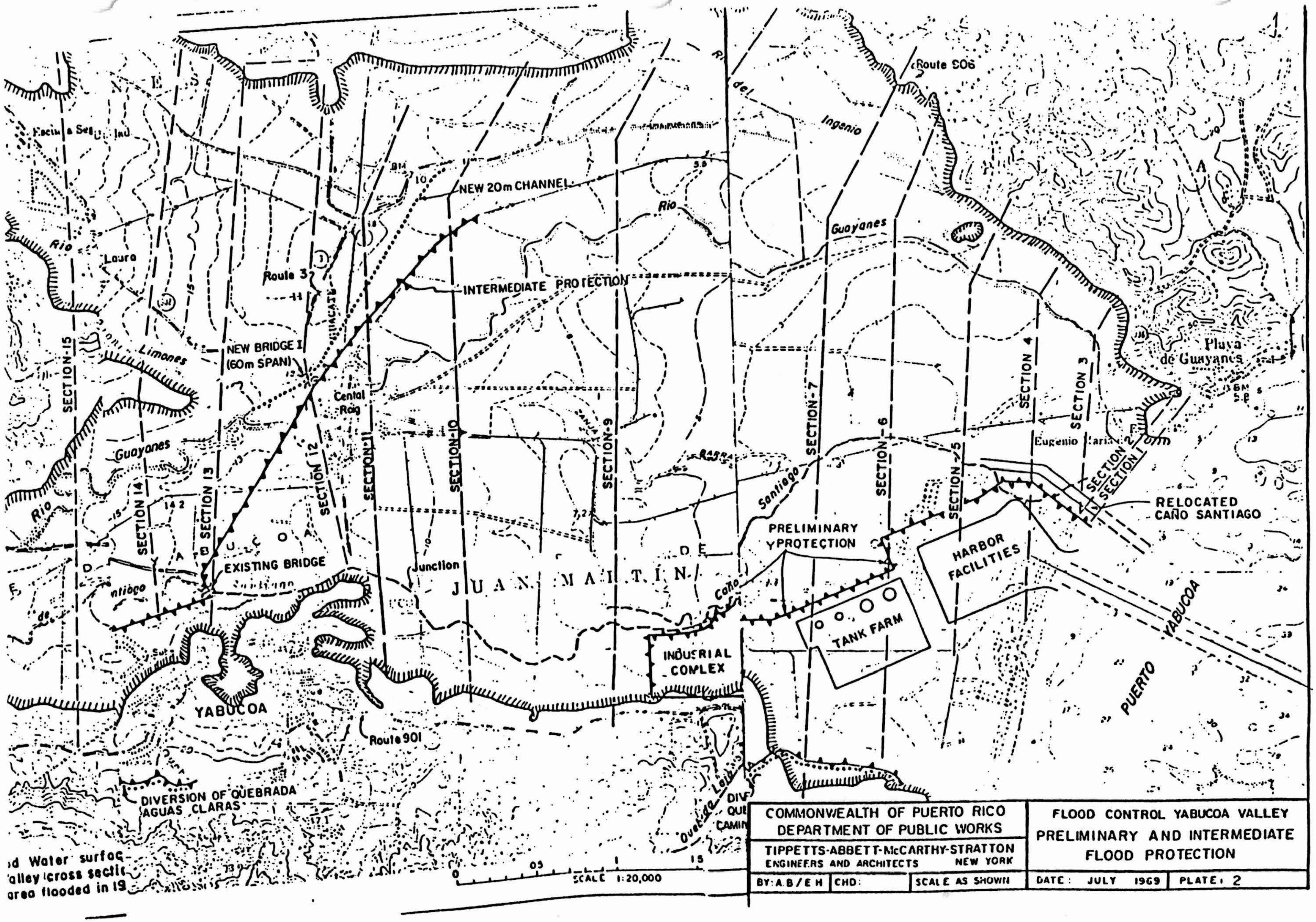
ENGINEERS AND ARCHITECTS NEW YORK
BY: E.H.
CHD: A B
SCALE AS SHOWN
FLOOD CONTROL YABUCOA VALLEY
FLOOD OF 1960 - WATER SURFACE
CONTOURS AND FLOOD LIMITS
(USGS DATA)
DATE: JULY 1969
PLATE: 1
SCALE 1:20,000
0
05
15
2 Km
urfaco conours
section -
1 in 1960
SECTION-1
SECTION-2
SECTION-3
SECTION-4
SECTION-5
SECTION-6
SECTION-7
SECTION-8
SECTION-9
SECTION-10
SECTION-11
SECTION-12
SECTION-13
SECTION-14
SECTION-15
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
Route 906
Route 3
Route 901
Rio del Ingenio
Rio Guayanés
Guayanés
Rio Limones
Laura
Central Roig
YABUCOA
VALLE DE JUAN MARTIN
VALLE DE YABUCOA
Caño de Santiago
Santiago
Playa de Guayanés
PUERTO YABUCOA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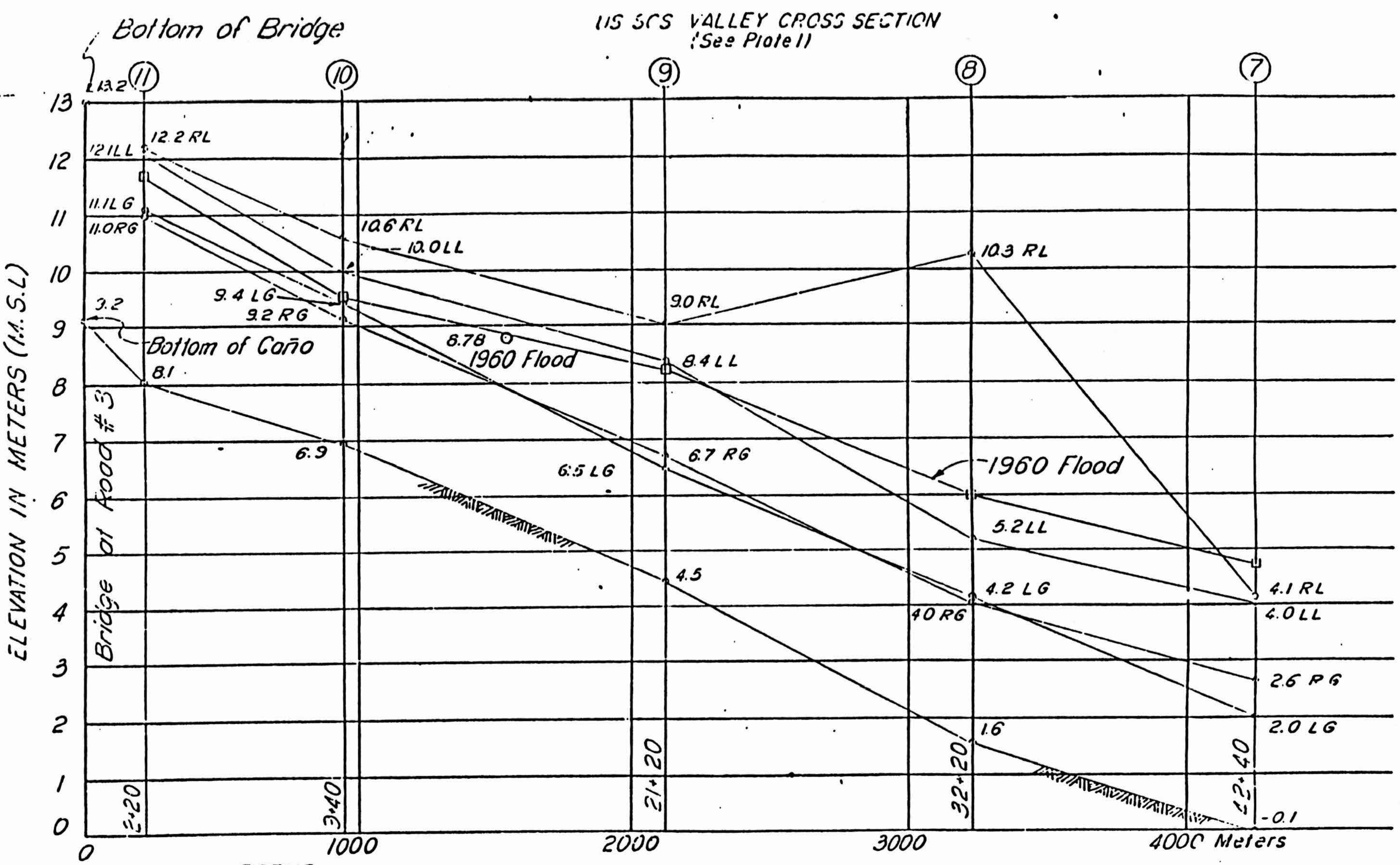
BY: A.B/E.H CHD: SCALE AS SHOWN
FLOOD CONTROL YABUCOA VALLEY
PRELIMINARY AND INTERMEDIATE
FLOOD PROTECTION
DATE: JULY 1969 PLATE: 2
SCALE 1:20,000
NEW 20m CHANNEL
INTERMEDIATE PROTECTION
PRELIMINARY PROTECTION
NEW BRIDGE I (60m SPAN)
EXISTING BRIDGE
INDUSTRIAL COMPLEX
TANK FARM
HARBOR FACILITIES
RELOCATED CAÑO SANTIAGO
DIVERSION OF QUEBRADA AGUAS CLARAS
YABUCOA
PUERTO YABUCOA
JUAN MARTIN
Playa de Guayanes
Route 3
Route 901
Route 906
Rio Guayanes
Rio del Ingenio
Santiago
Limones
Laura
Cental Roig
Eugenio Maria
Junction
SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5
SECTION 6
SECTION 7
SECTION 9
SECTION 10
SECTION 11
SECTION 12
SECTION 13
SECTION 14
SECTION 15



LEGEND

RL - Right Levee (Existing)
LL - Left Levee (Existing)
LG - Left Ground (Existing)
RG - Right Ground (Existing)
—○— Computed Water Elevation
○ Flood Mark (1960)

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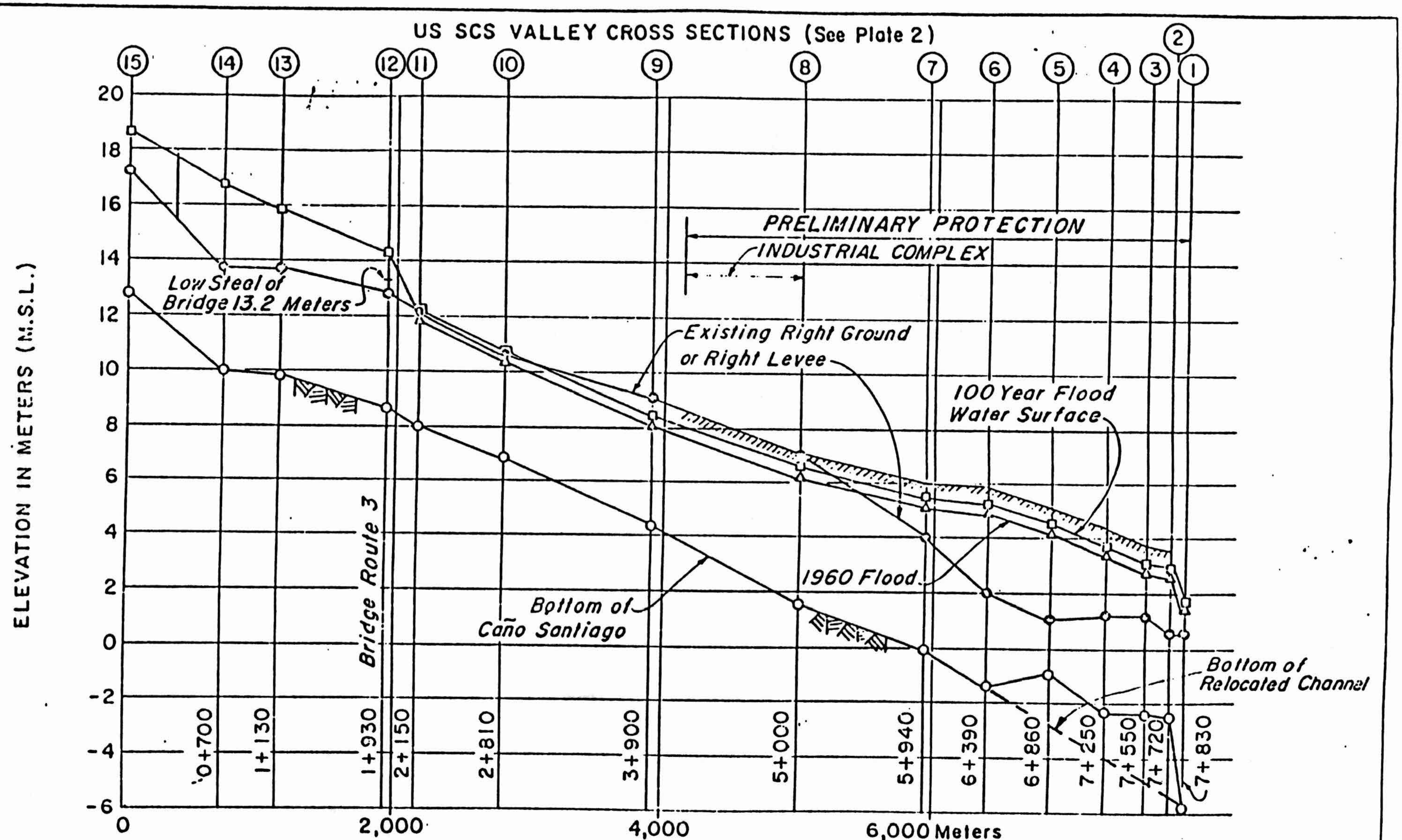
FLOOD CONTROL YABUCOA VALLEY
CAÑO SANTIAGO PROFILE
PRESENT CONDITIONS

BY: CH/AB	CHD: AB	SCALE: H 1:20,000 V. 1:100	DATE: JULY 1969	PLATE: 3



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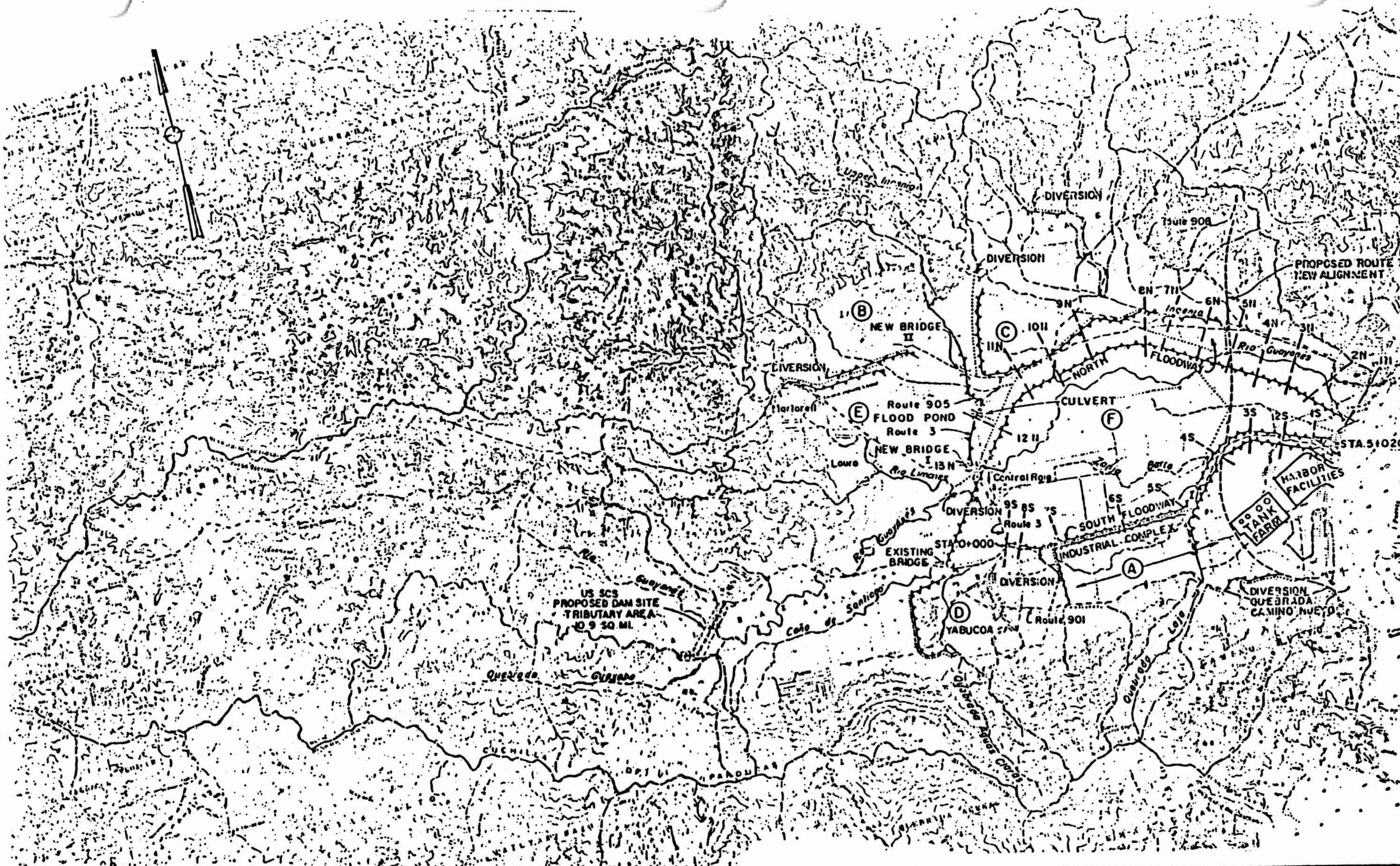
ENGINEERS AND ARCHITECTS NEW YORK

FLOOD CONTROL YABUCOA VALLEY
CAÑO SANTIAGO PROFILE
PRELIMINARY PROTECTION

BY: FH/AB | CHD: AB | SCALE: H;1:40,000

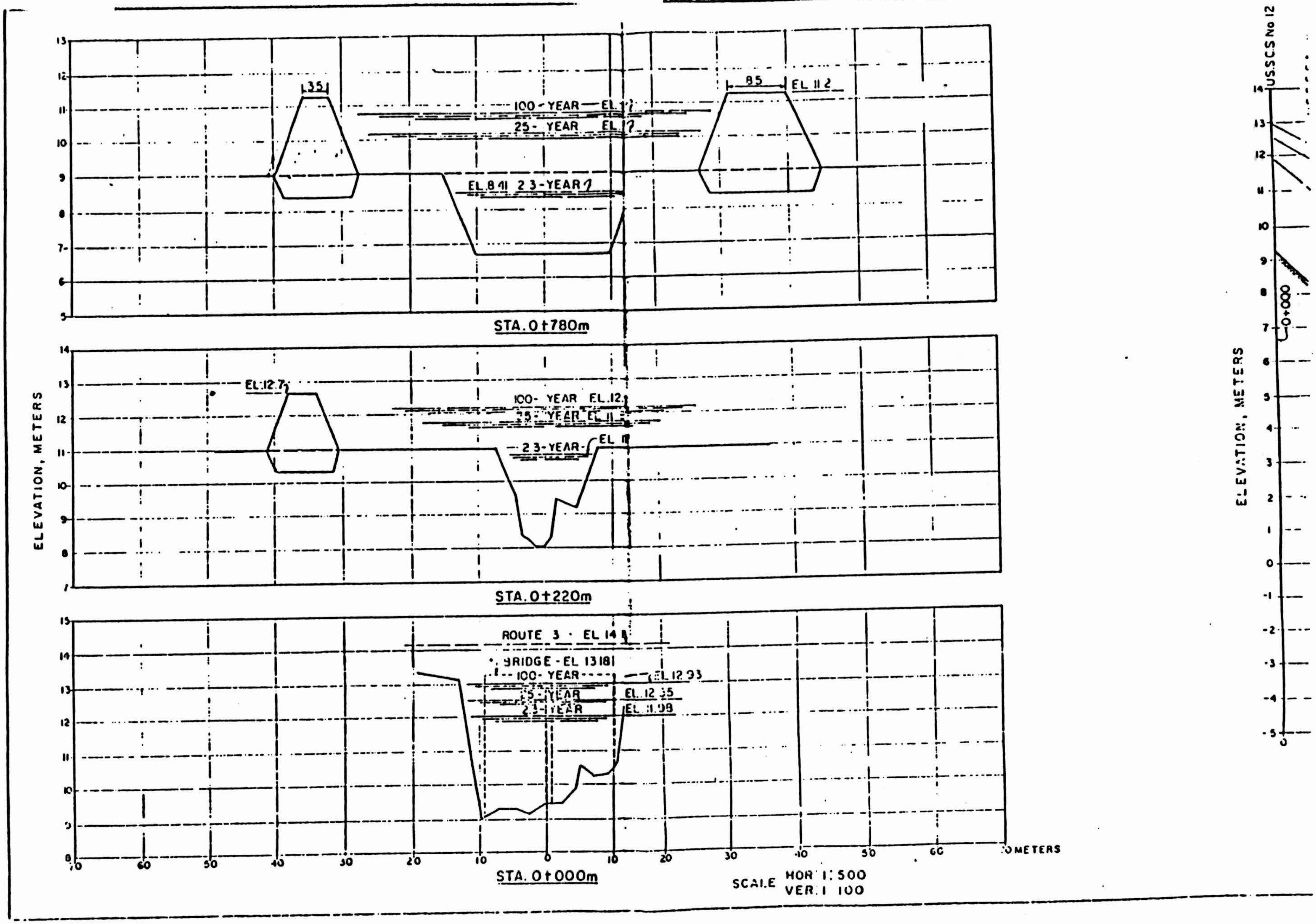


LEGEND:
- .... Relocated Channel
- ——— Natural Channel
- —·—·— Boundary of Tributary area
- Levee
- (D) Protected Area

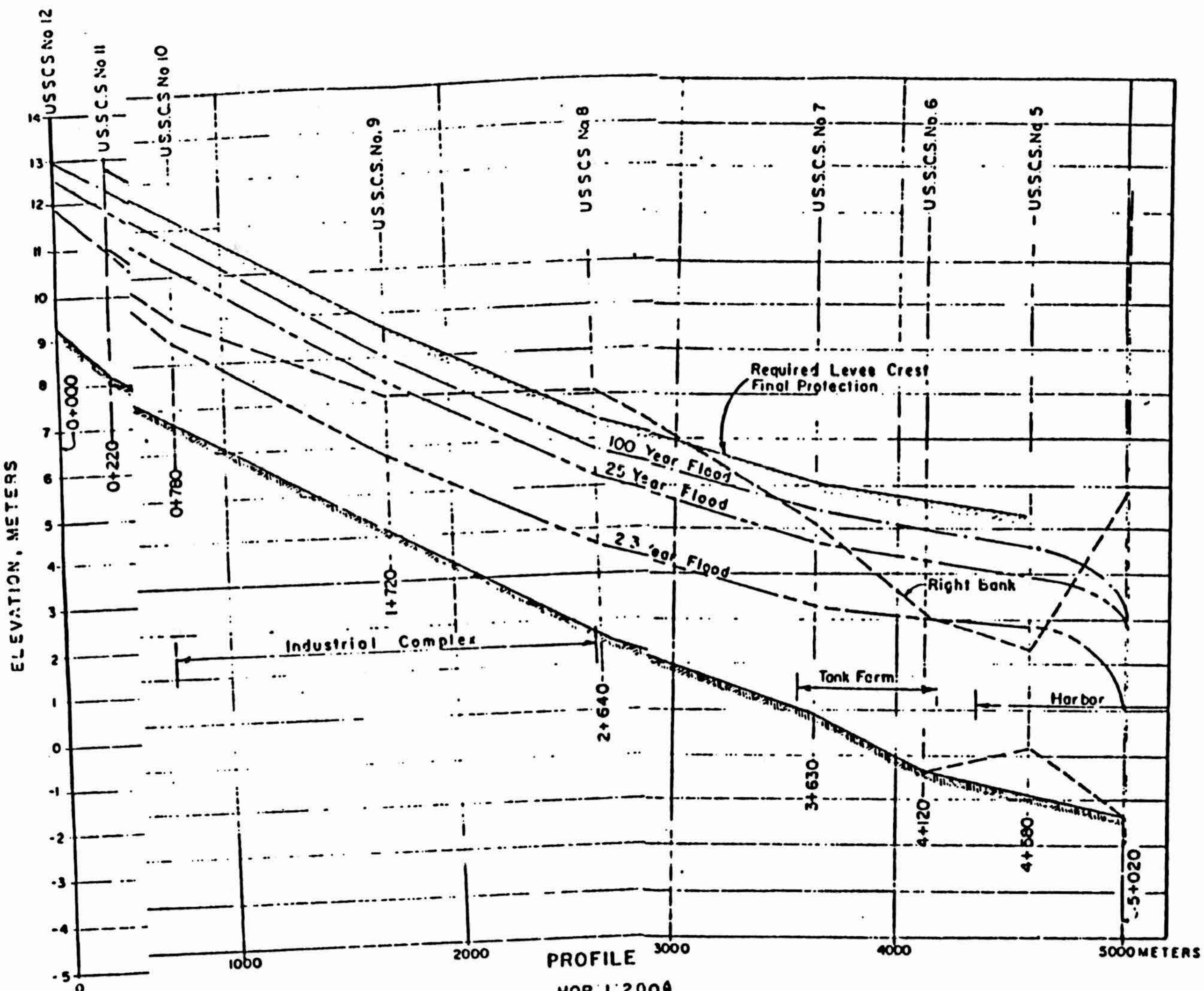
0 1 2 3 4 Km

SCALE 1: 30,000

COMMONWEALTH OF PUERTO RICO DEPARTMENT OF PUBLIC WORKS			FLOOD CONTROL YABUCOA VALLEY PROTECTION PLAN FOR FINAL INDUSTRIAL DEVELOPMENT	
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK				
BY EH/AB	CHD. A.B.	SCALE: AS SHOWN	DATE: JULY 1969	PLATE: 5



STA. 0+780m
100-YEAR EL 11
25-YEAR EL 1?
EL 8.41 2.3-YEAR
8.5
EL 11.2
3.5
STA. 0+220m
EL 12.7
100-YEAR EL 12
25-YEAR EL 11
2.3-YEAR EL 11
STA. 0+000m
ROUTE 3 · EL 14
BRIDGE - EL 13.181
100-YEAR EL 12.93
25-YEAR EL 12.35
2.3-YEAR EL 11.98
ELEVATION, METERS
METERS
SCALE HOR 1:500
VER. 1:100
US.SCS No 12
0+000



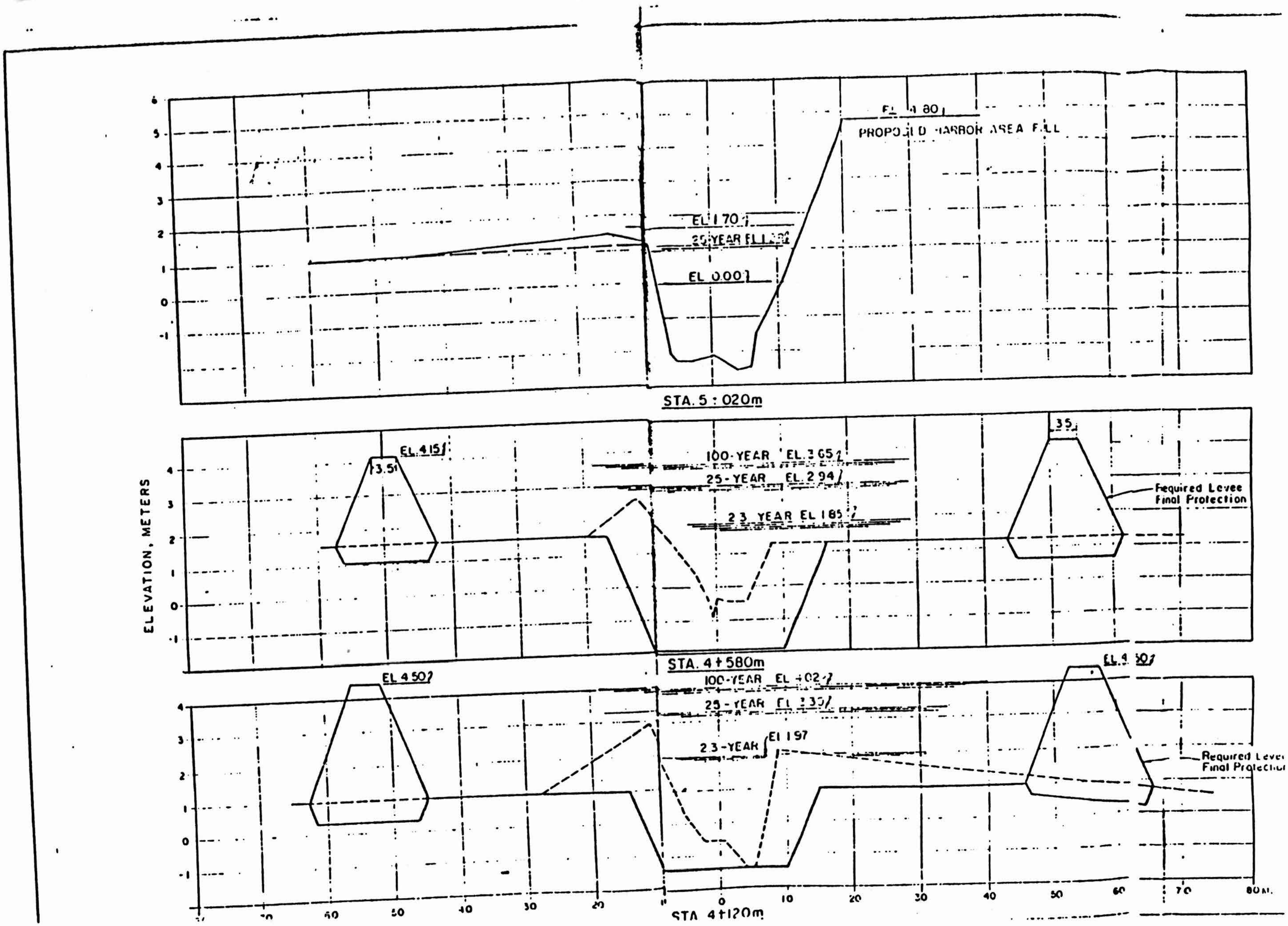
PROFILE

SCALE HOR. 1:2000
VER. 1:100

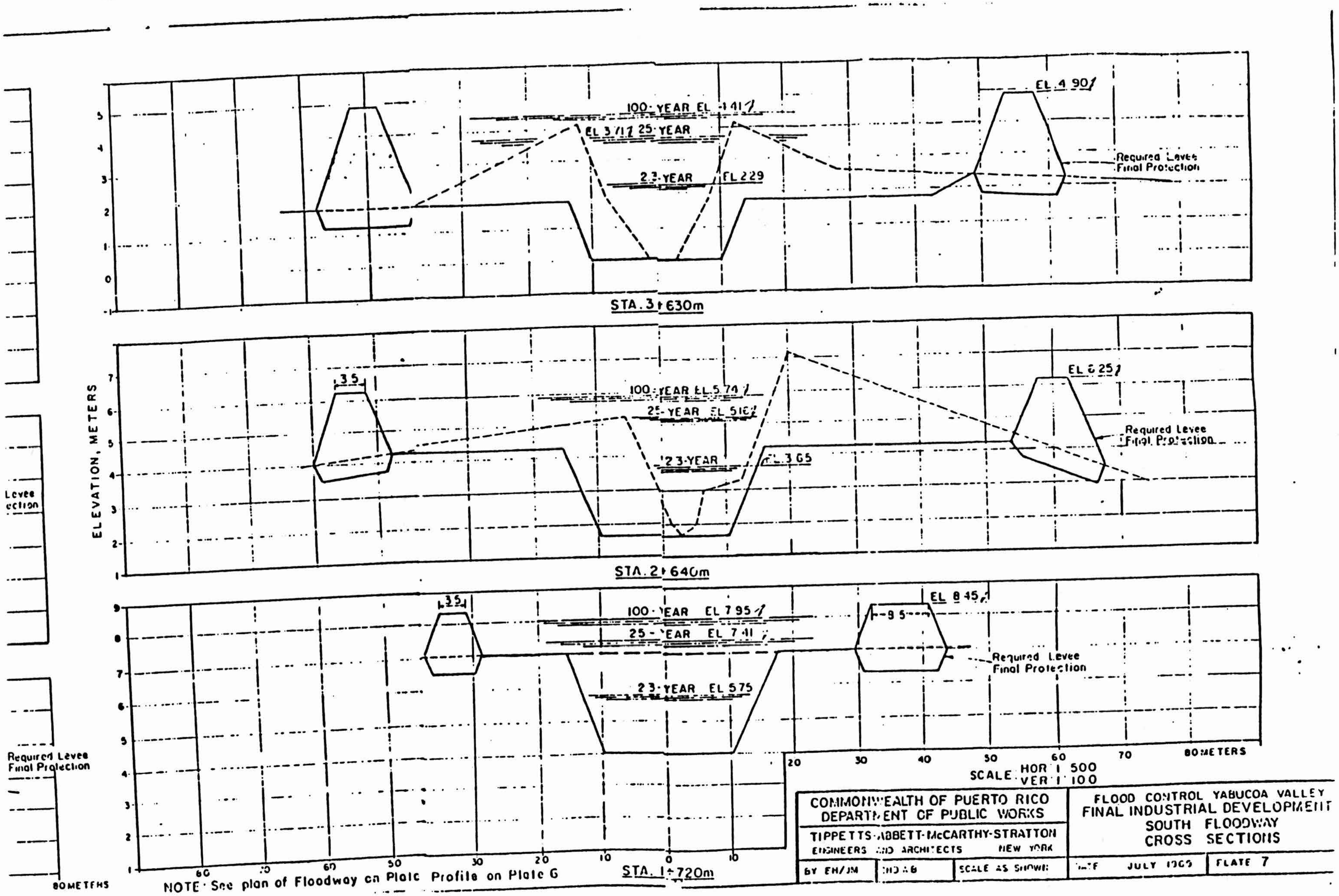
NOTE:
See plan of Floodway on Plate 5

METERS

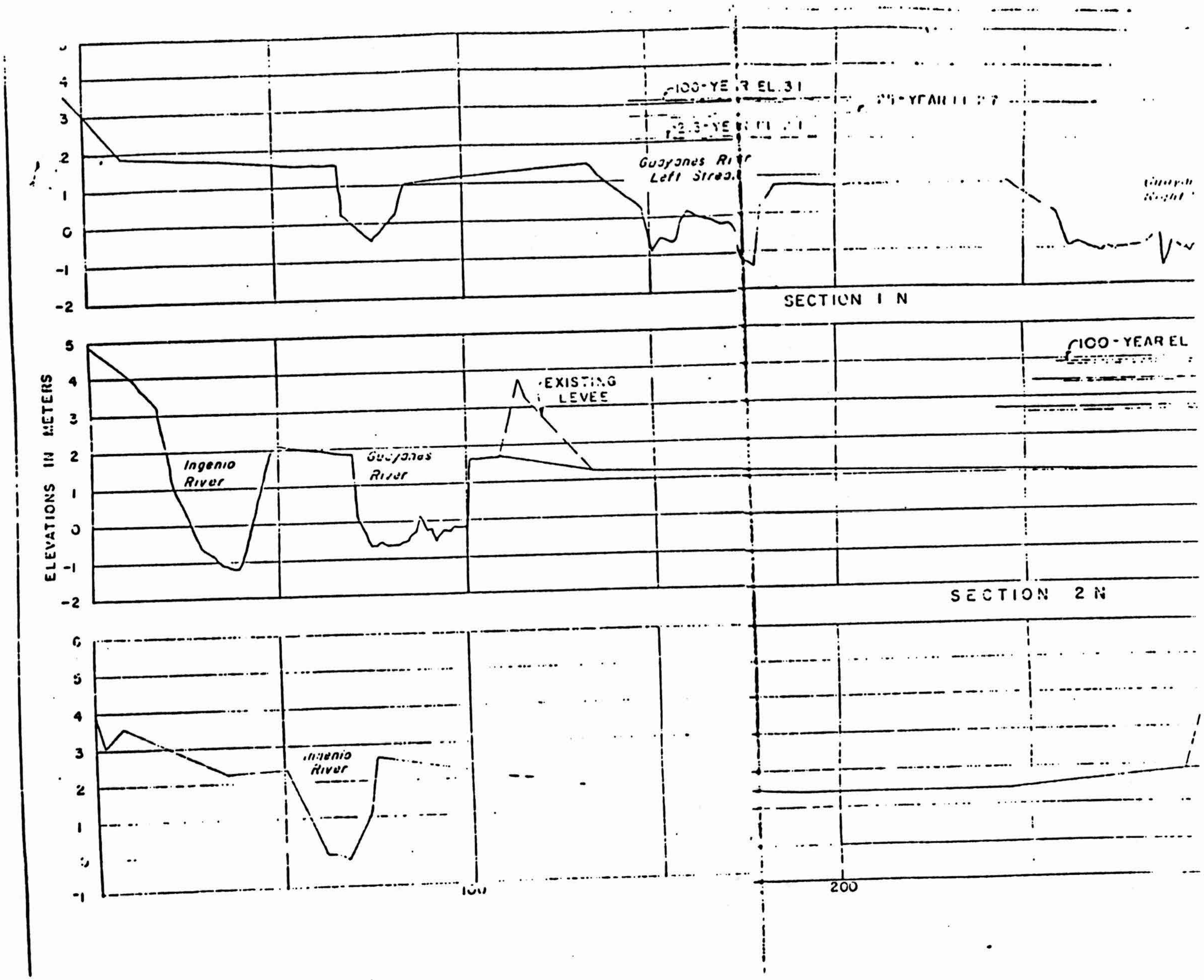
COMMONWEALTH OF PUERTO RICO DEPARTMENT OF PUBLIC WORKS			FLOOD CONTROL YABUCOA VALL[EY] FINAL INDUSTRIAL DEVELOPM[ENT] SOUTH FLOODWAY PROFILE AND CROSS SECTI[ON]	
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK				
BY EH/JM	CHD. AB	SCALE AS SHOWN	DATE JULY 1969	PLATE 6

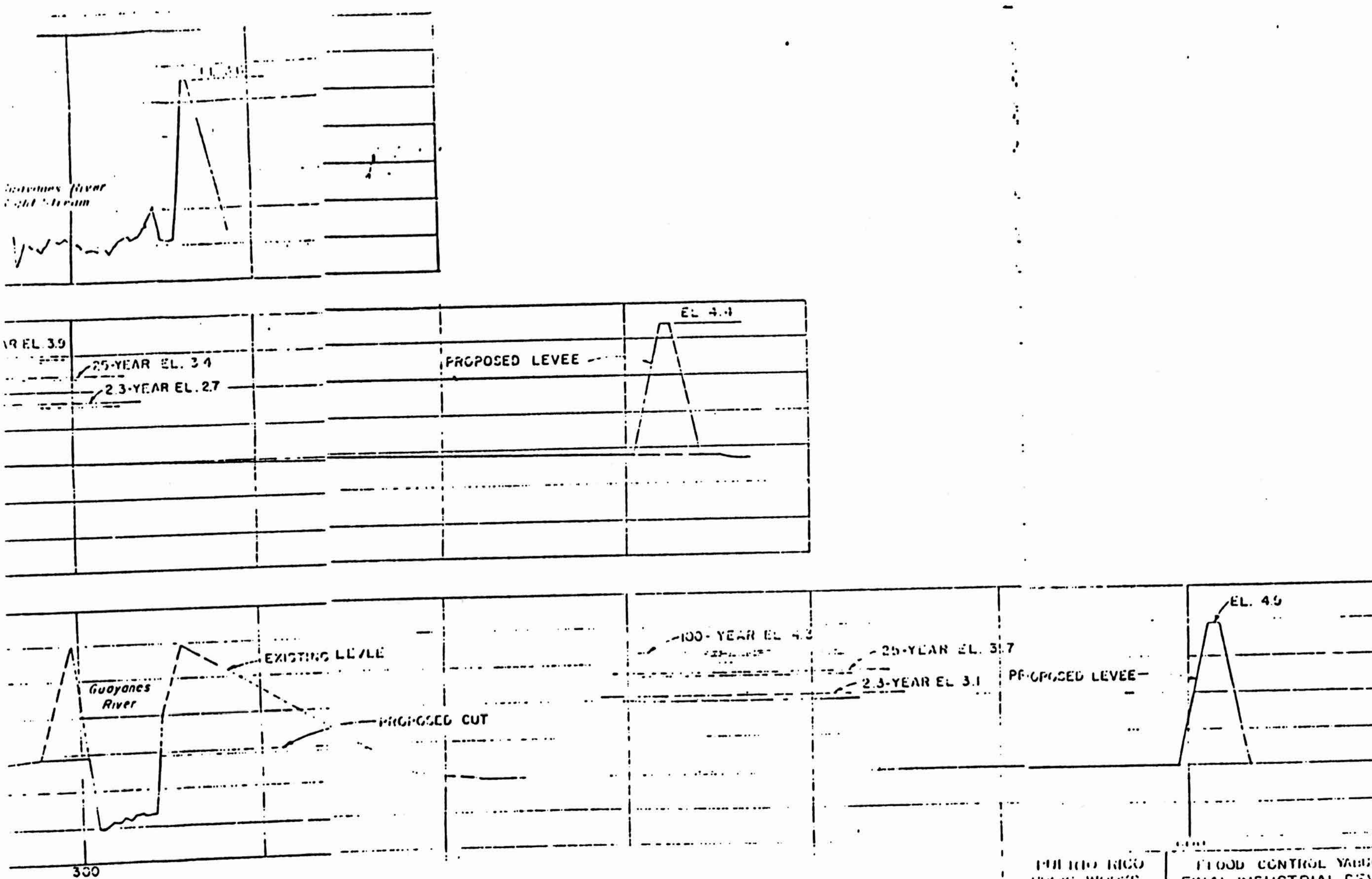


EL 4.80
PROPOSED HARBOR AREA FILL
EL 1.70
25-YEAR EL
EL 0.00
STA. 5 : 020m
EL 4.15
3.51
3.5
100-YEAR EL 3.65
25-YEAR EL 2.94
2.3 YEAR EL 1.85
Required Levee
Final Protection
ELEVATION, METERS
STA. 4+580m
EL 4.50
EL 4.50
100-YEAR EL 4.02
25-YEAR EL 3.30
2.3-YEAR EL 1.97
Required Levee
Final Protection
STA. 4+120m



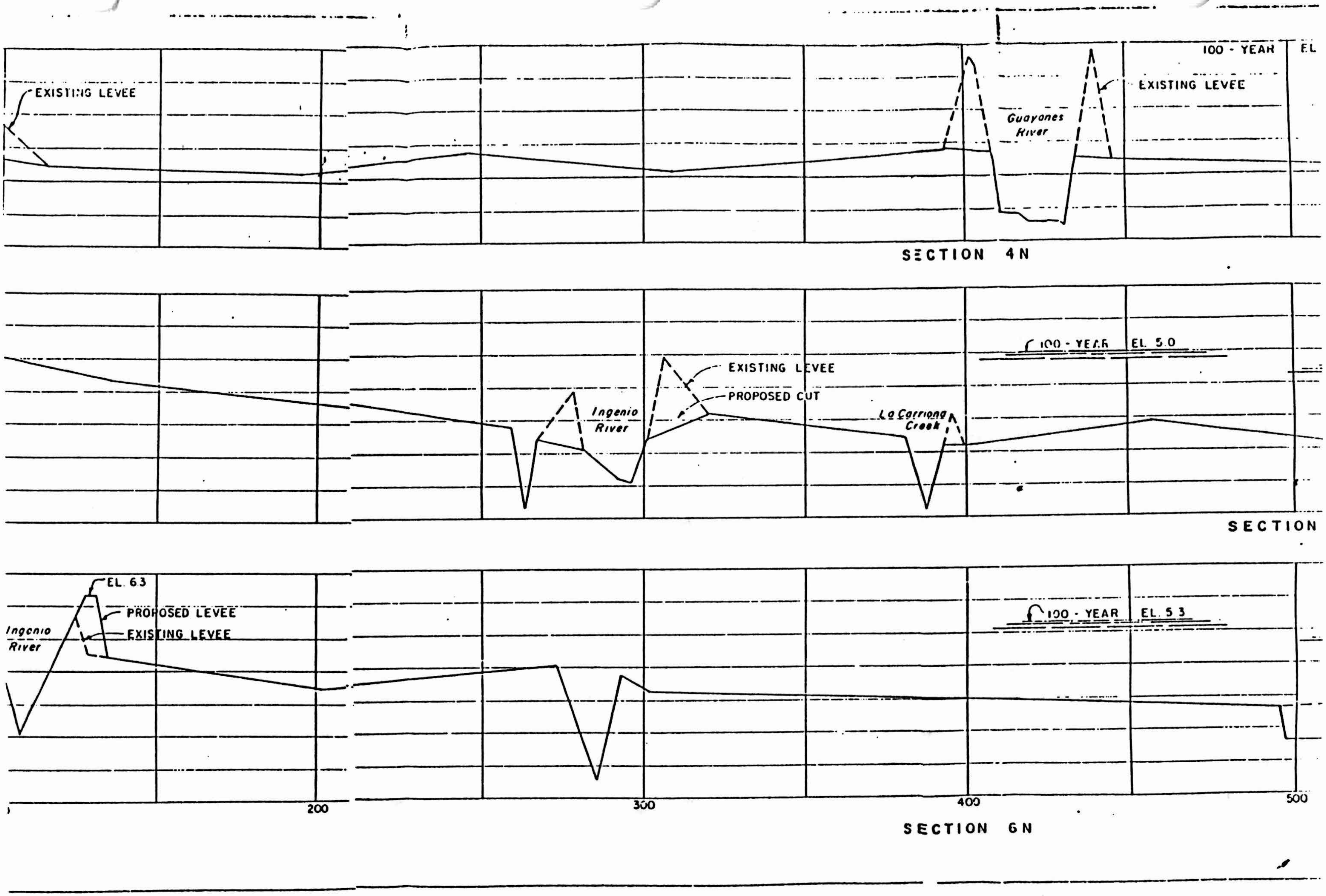
COMMONWEALTH OF PUERTO RICO DEPARTMENT OF PUBLIC WORKS	FLOOD CONTROL YABUCOA VALLEY FINAL INDUSTRIAL DEVELOPMENT SOUTH FLOODWAY CROSS SECTIONS
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK	
BY EH/JM · CHD A.B · SCALE AS SHOWN	DATE JULY 1969 · PLATE 7


ELEVATIONS IN METERS
100-YEAR EL.3.1
Guayanes River Left Stream
SECTION 1 N
100-YEAR EL
EXISTING LEVEE
Ingenio River
Guayanes River
SECTION 2 N
Ingenio River
100
200

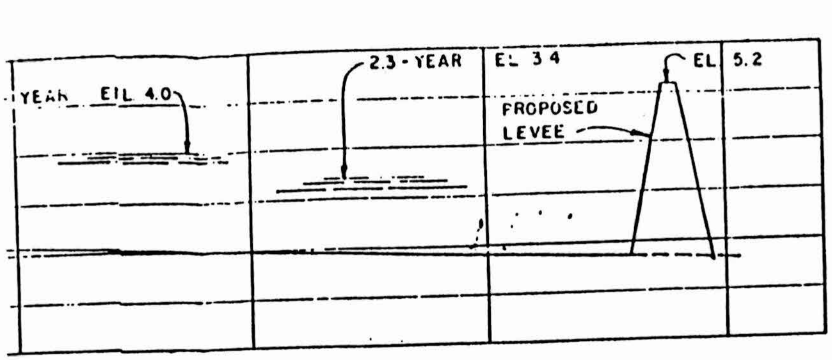


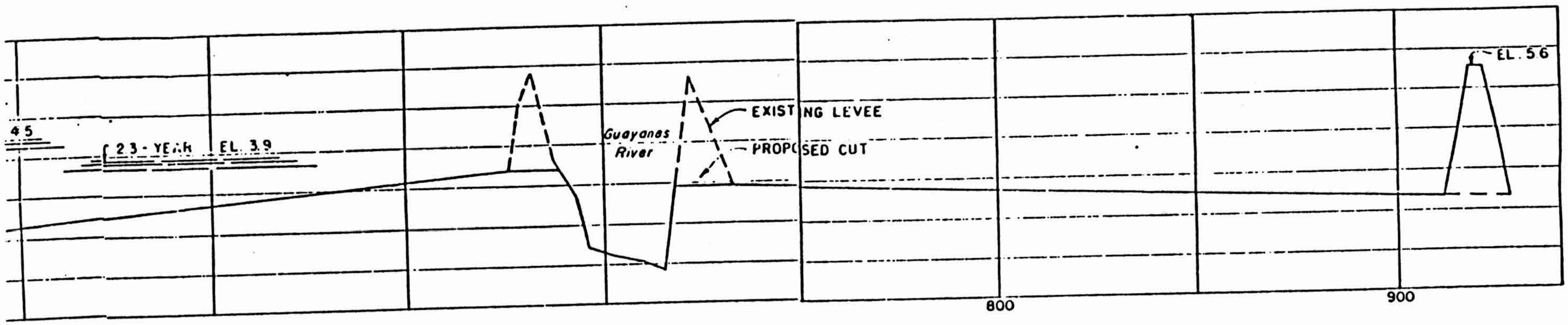
SECTION 3N

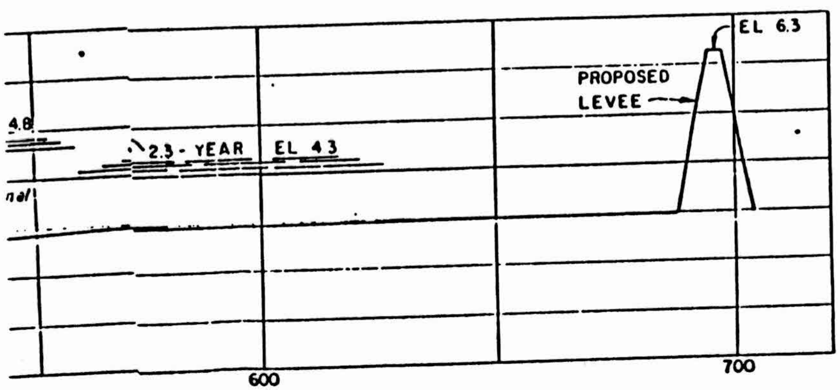
PUERTO RICO PUBLIC WORKS	FLOOD CONTROL YABUCOA VALLEY FINAL INDUSTRIAL DEVELOPMENT NORTH FLOODWAY CROSS SECTIONS	
SCALE AS SHOWN	DATE JULY 1965	PLATE 3



100 - YEAR EL
EXISTING LEVEE
EXISTING LEVEE
Guayones River
SECTION 4N
100 - YEAR EL. 5.0
EXISTING LEVEE
PROPOSED CUT
Ingenio River
La Carriona Creek
SECTION
EL. 63
PROPOSED LEVEE
EXISTING LEVEE
Ingenio River
100 - YEAR EL. 5.3
200
300
400
500
SECTION 6N



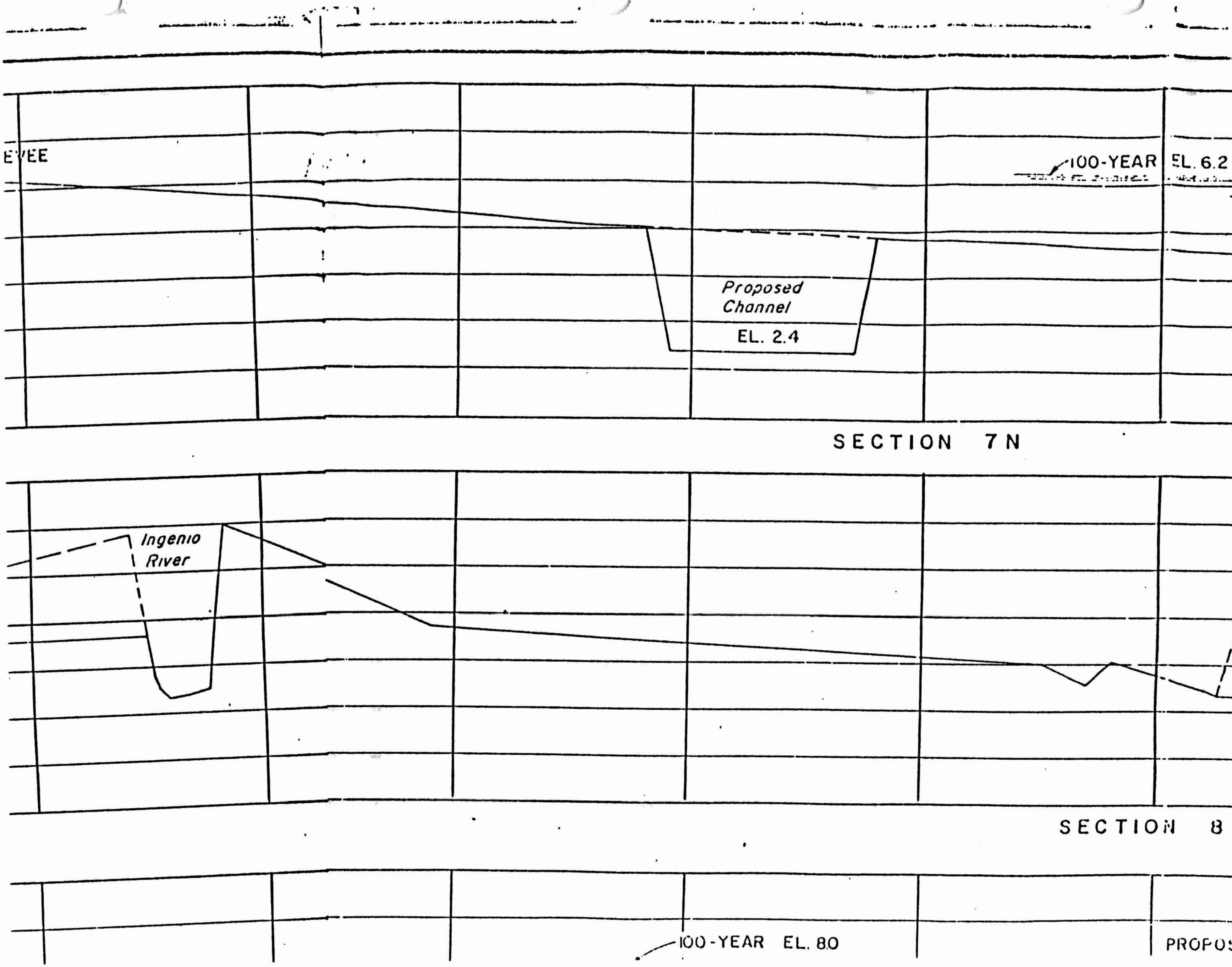




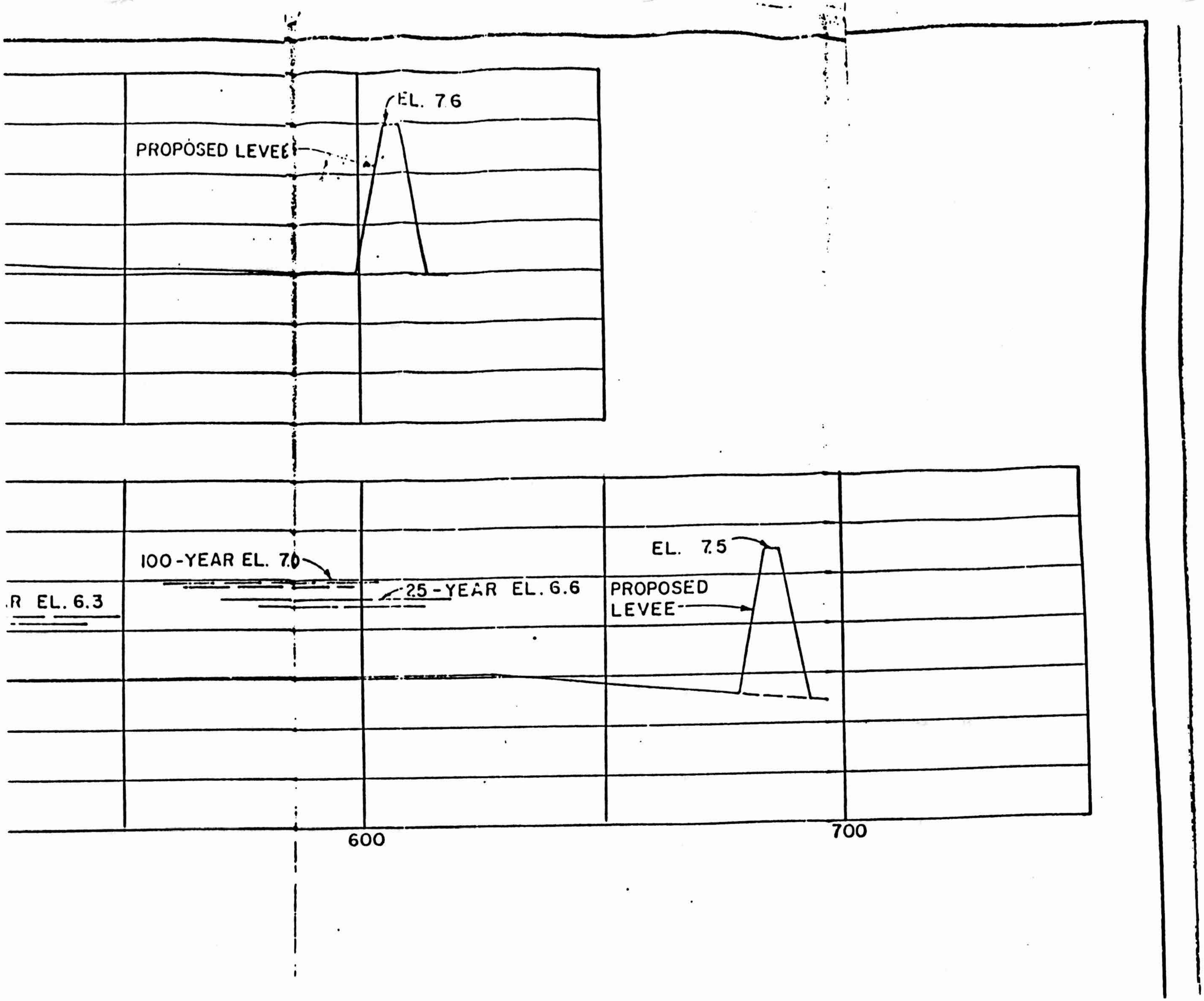
SCALE: HORIZ 1 : 1000 M
VERT 1 : 100 M

COMMONWEALTH OF PUERTO RICO DEPARTMENT OF PUBLIC WORKS	FLOOD CONTROL YABUCOA VALLEY FINAL INDUSTRIAL DEVELOPMEN NORTH FLOODWAY PROFILE AND CROSS SECTIONS
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK	

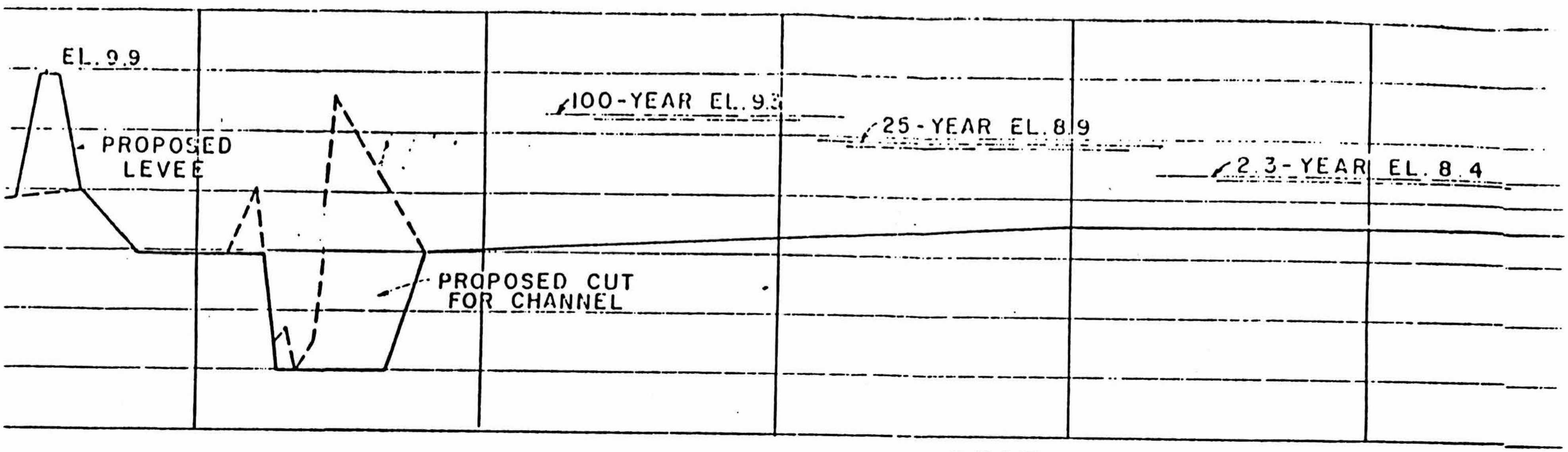
BY EH/JM	CHD A B	SCALE AS SHOWN	DATE JULY 1969	PLATE 9



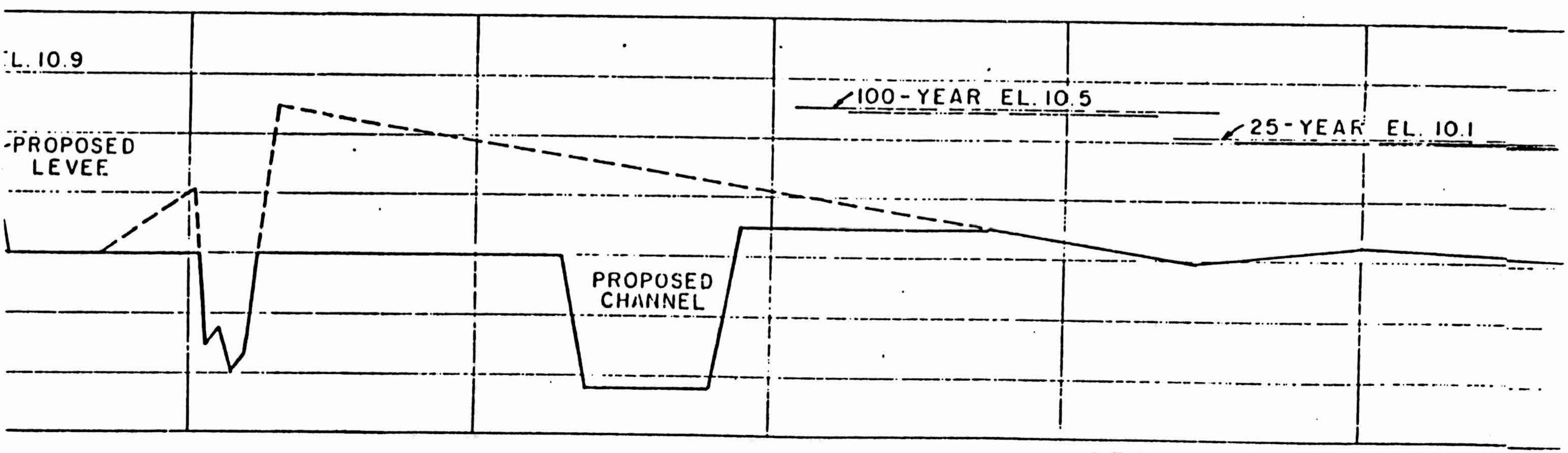
E'VEE
100-YEAR EL. 6.2
Proposed Channel
EL. 2.4
SECTION 7N
Ingenio River
SECTION 8
100-YEAR EL. 8.0
PROPOS



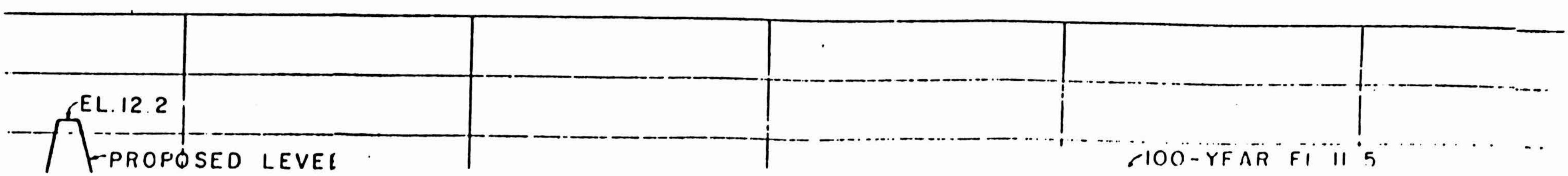
EL. 7.6
PROPOSED LEVEE
100-YEAR EL. 7.0
25-YEAR EL. 6.6
R EL. 6.3
EL. 7.5
PROPOSED LEVEE
600
700

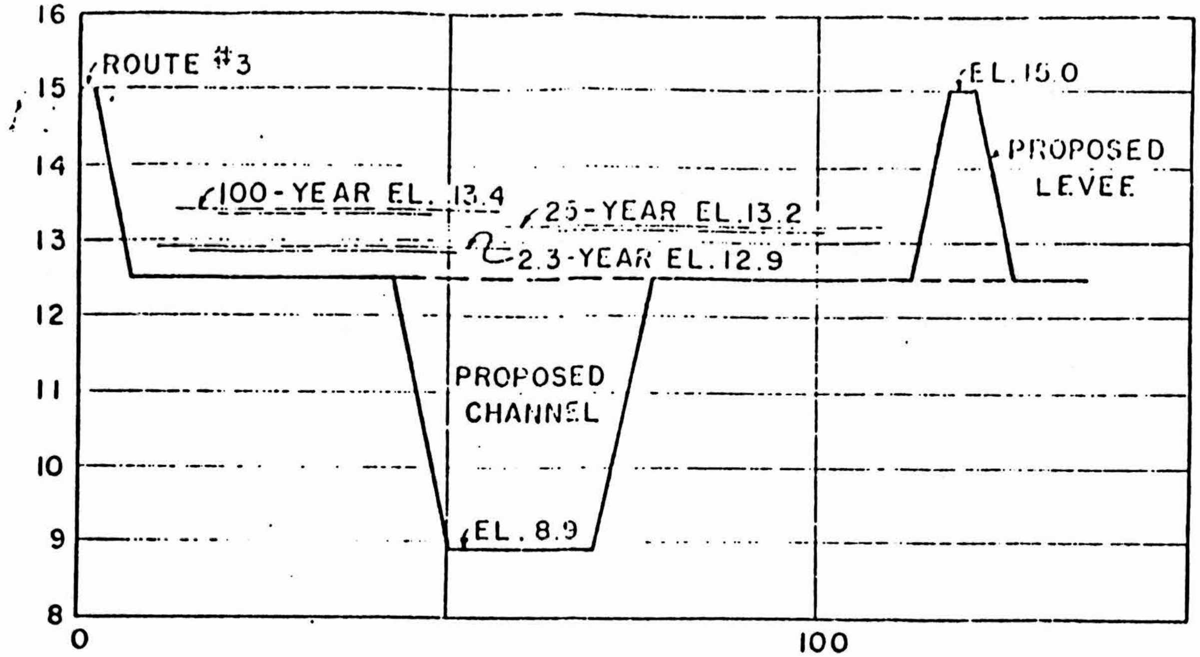


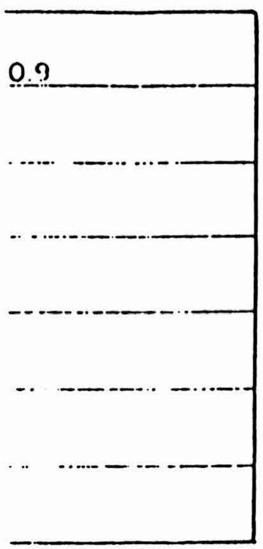
SECTION 10 N

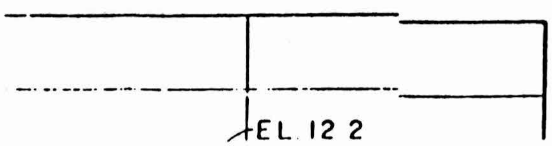


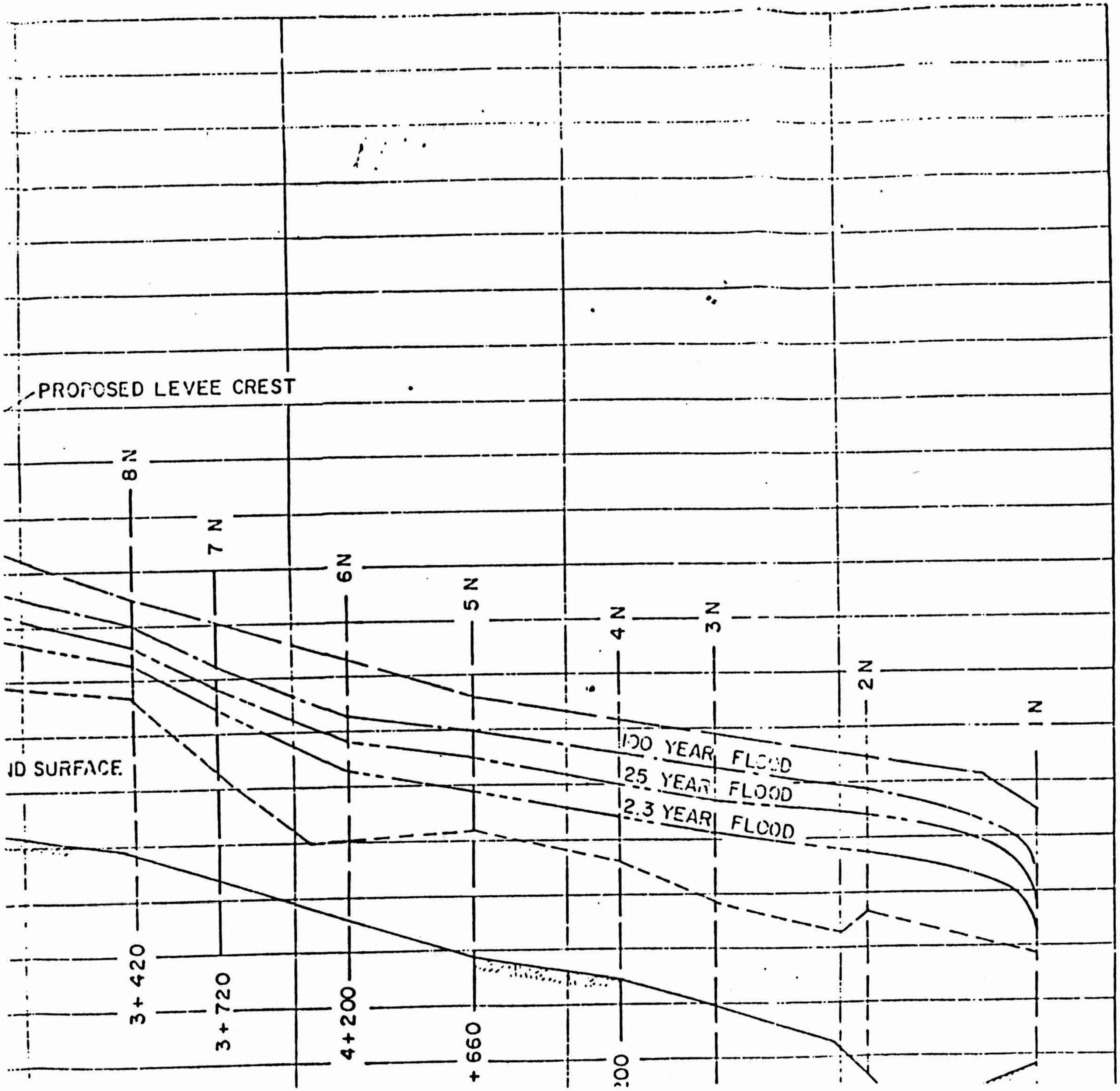
SECTION 11 N




ROUTE #3
EL. 15.0
PROPOSED
LEVEE
100-YEAR EL. 13.4
25-YEAR EL. 13.2
2.3-YEAR EL. 12.9
PROPOSED
CHANNEL
EL. 8.9
16
15
14
13
12
11
10
9
8
0
100
SECTION 13N


0.9


EL. 12 2



PROPOSED LEVEE CREST
8 N
7 N
6N
5 N
4 N
3 N
2 N
N
100 YEAR FLOOD
25 YEAR FLOOD
2.3 YEAR FLOOD
ID SURFACE
3 + 420
3 + 720
4 + 200
+ 660

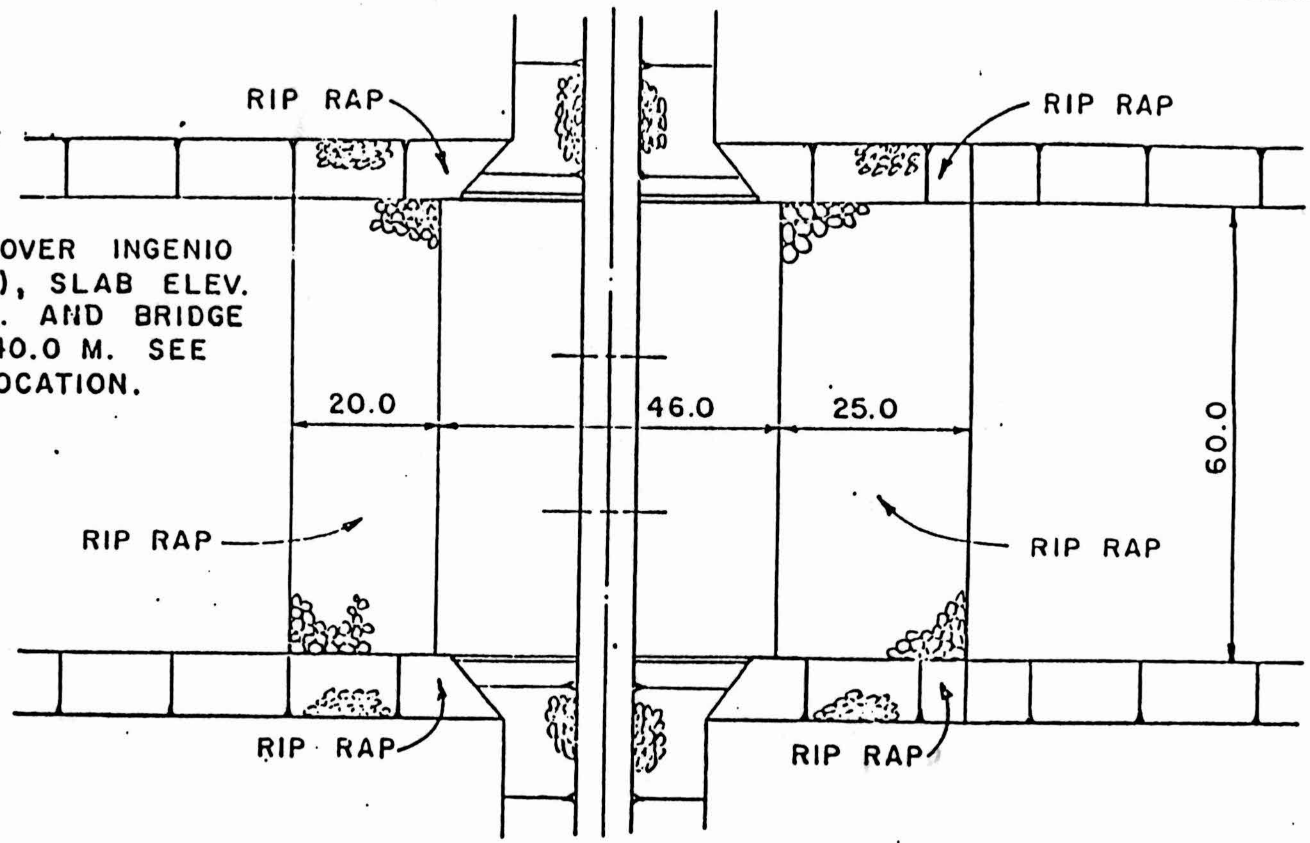
COMMONWEALTH OF PUERTO RICO
DEPARTMENT OF PUBLIC WORKS

TIPPETTS-ABBETT-McCARTHY-STRATTON
ENGINEERS AND ARCHITECTS NEW YORK

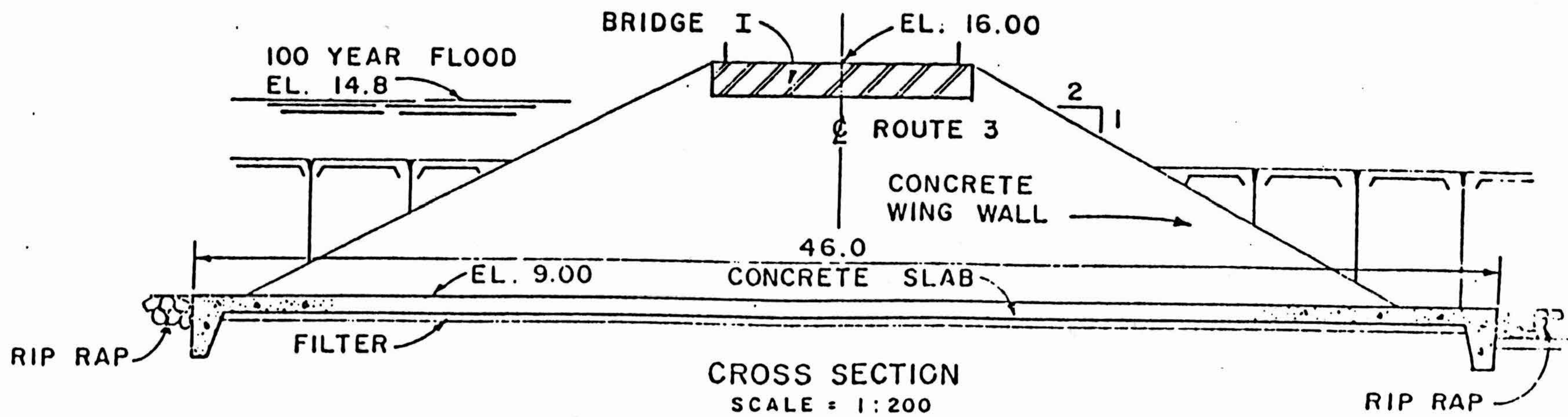
FLOOD CONTROL YABUCOA VALLEY
FINAL INDUSTRIAL DEVELOPMENT
CONTROL STRUCTURE CROSSING OF RT. 3
OVER RELOCATED GUAYANES RIVER

NOTE:

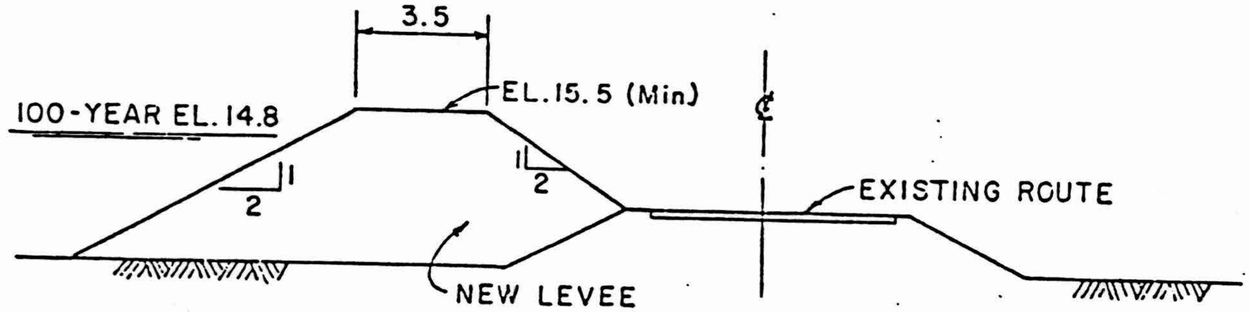
FOR STRUCTURE OVER INGENIO RIVER (BRIDGE II), SLAB ELEV. WILL BE 9.50 M. AND BRIDGE SPAN WILL BE 40.0 M. SEE PLATE 5 FOR LOCATION.



PLAN
SCALE = 1 : 100



CROSS SECTION
SCALE = 1 : 200



CROSS SECTION

COMMONWEALTH OF PUERTO RICO DEPARTMENT OF PUBLIC WORKS	FLOOD CONTROL YABUCOA VALLEY FINAL INDUSTRIAL DEVELOPMENT
TIPPETTS-ABBETT-McCARTHY-STRATTON ENGINEERS AND ARCHITECTS NEW YORK	LEVEES FOR PROPOSED PONDAGE AREA

BY: E H./J.M.	CHD: A. B.	SCALE: 1:20	DATE: [illegible]	[illegible]